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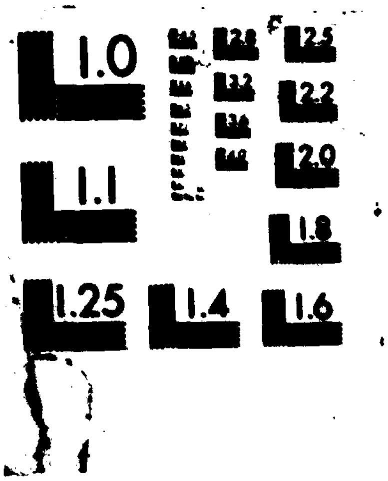
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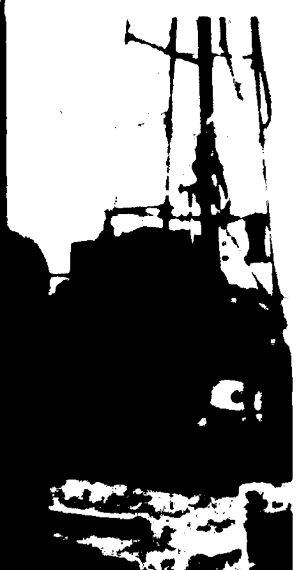

by

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Western Division
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Eleven sites at the Naval Weapons Station, Concord, California, have been investigated for suitability for hazardous waste disposal. State and Federal regulations and technical guidance on siting and on disposal practice in general were used as the criteria for evaluating each site. Five sites were rejected as unsuitable based on surficial examinations. Bedrock exposed at two sites was regarded as unacceptable by virtue of the likelihood of adverse secondary permeability along rock joints. Potential for slope instability largely eliminated a third site, and high water table made two others unsatisfactory for meeting state requirements.

Five remaining sites passed the early screening and required subsurface investigations for completion of the evaluation. Both field tests and laboratory tests were undertaken to determine the coefficient of permeability. The coefficient ranged from 10^{-3} to
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10^{-8} cm/sec at the small scale of laboratory testing. Field-determined values mostly ranged more narrowly from 10^{-5} through 10^{-6} cm/sec in bracketing a variety of material types over vertical intervals of several feet.

None of the eleven sites strictly satisfied State criteria for low permeability.

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PREFACE

Field and laboratory investigations were conducted by the US Army Engineer Waterways Experiment Station (WES) at the Naval Weapons Station, Concord, California (NWS Concord), from February through May 1987. The Geotechnical Laboratory (GL) undertook this work for the Naval Facilities Engineering Command to evaluate suitability of sites for disposal of hazardous substances presently situated in the northeastern corner of NWS Concord. This work constitutes a portion of a comprehensive feasibility study of contamination remediation under way at WES. The South Pacific Division Laboratory (SPDL), Corps of Engineers, conducted the laboratory testing.

This report was prepared by Dr. R. J. Lutton, Engineering Geology and Rock Mechanics Division (EGRMD), who also served as Principal Investigator, Messers R. D. Bennett and C. C. McAneny of EGRMD and D. N. Wong of SPDL. The work was initiated through the Site Characterization Unit of which J. H. May is Chief. This report was prepared under the general supervision of Drs. D. C. Banks, Chief, EGRMD, and W. F. Marcuson III, Chief, GL. Ms. Odell F. Allen, Information Products Division, Information Technology Laboratory, edited the report.

COL Dwayne G. Lee, CE, was Commander and Director of WES during the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (force) per square foot	95.76052	kilopascals

SUITABILITY OF SITES FOR HAZARDOUS WASTE DISPOSAL

CONCORD NAVAL WEAPONS STATION

CONCORD, CALIFORNIA

PART I: INTRODUCTION

Purpose

1. The purpose of this investigation was to determine the suitability of sites at the Naval Weapons Station (NWS), Concord, Calif., as facilities for disposal of hazardous waste. This report presents the pertinent findings and conclusions of the investigation.

Scope

2. This report evaluates the suitability of sites on NWS Concord identified as potential sites for the disposal of hazardous substances presently located on Parcels 572, 573, 574, 575, 576, 579D, and 581 on NWS Concord. Evaluations were made with reference to criteria established by the State of California Water Quality Control Board and Department of Health Services and the US Environmental Protections Agency. The report summarizes both the preliminary and the intermediate phases of site exploration conducted in the investigation. If any of the sites studied is chosen for the development of a landfill for the disposal of the hazardous substances, a third phase of additional field investigations will be required. Those future studies would focus on site characterization with more drilling, sampling, and testing as well as documentation of geological details in three dimensions.

3. Eleven sites identified as potential sites for landfills by the Naval Facilities Engineering Command, Western Division, were evaluated against the suitability criteria in the preliminary screening phase. The criteria are described in Part II. The preliminary screening evaluations are described in Part III. The sites surviving the screening were subjected to field investigations, which concentrated on the determination of important permeability characteristics. Field and laboratory methods are described in Parts IV and V. The evaluations of the five sites investigated further in the field are

presented in Part VI, and the conclusions made from these and the preliminary evaluations and analyses are summarized in Part VII.

Geology and Hydrology

4. The geology and hydrology of an area constitute important background to siting waste-disposal facilities. The regional setting often presents important constraints or limitations on potential sites. Near the NWS Concord, the salient features are steepsided hills composed mostly of Tertiary sedimentary rocks and broad lowlands floored with thick deposits of Quaternary alluvium (Figure 1). The wetlands in the northern part of the region along Suisun Bay are underlain by estuarine and riverine deposits.

5. The oldest formations are Tertiary sedimentary rocks exposed in Los Medanos Hills along the east side of NWS Concord. Figure 1 shows the extent of rock formations not only in Los Medanos Hills but also in smaller exposures to the southwest. The rock formations are composed of interbedded units of sandstone, siltstone, and shale. Sandstone is most resistant and conspicuous in outcrop and seems to constitute more than half the rock formations. Beds are typically steeply inclined. Jointing has combined with parting along bedding to facilitate weathering and development of boulder blocks in residual soil. The residual soil is susceptible to landsliding.

6. Alluvium in areas other than the hills can be categorized into two types on the basis of physical characteristics. One variety is somewhat stiffer or firmer than the other. This distinction probably reflects a difference in age with the firmer alluvium having been previously buried and therefore relatively old. Figure 1 shows the firmer alluvium mainly as a band of low hills situated centrally within Clayton Valley. The old alluvium may be roughly equivalent to beds mapped northeast of Suisun Bay as the Montezuma formation. The greater age of this alluvium is also manifested structurally. The band of hills has been mapped as an anticline with bedding dips of several degrees away from the topographic crest. This older alluvium probably correlates broadly with alluvium found at depths of several hundred feet in water wells north and south of Suisun Bay, along Clayton Valley, and within the city limits of Concord.

7. Both types of alluvium consist of beds of sandy, silty, and clayey

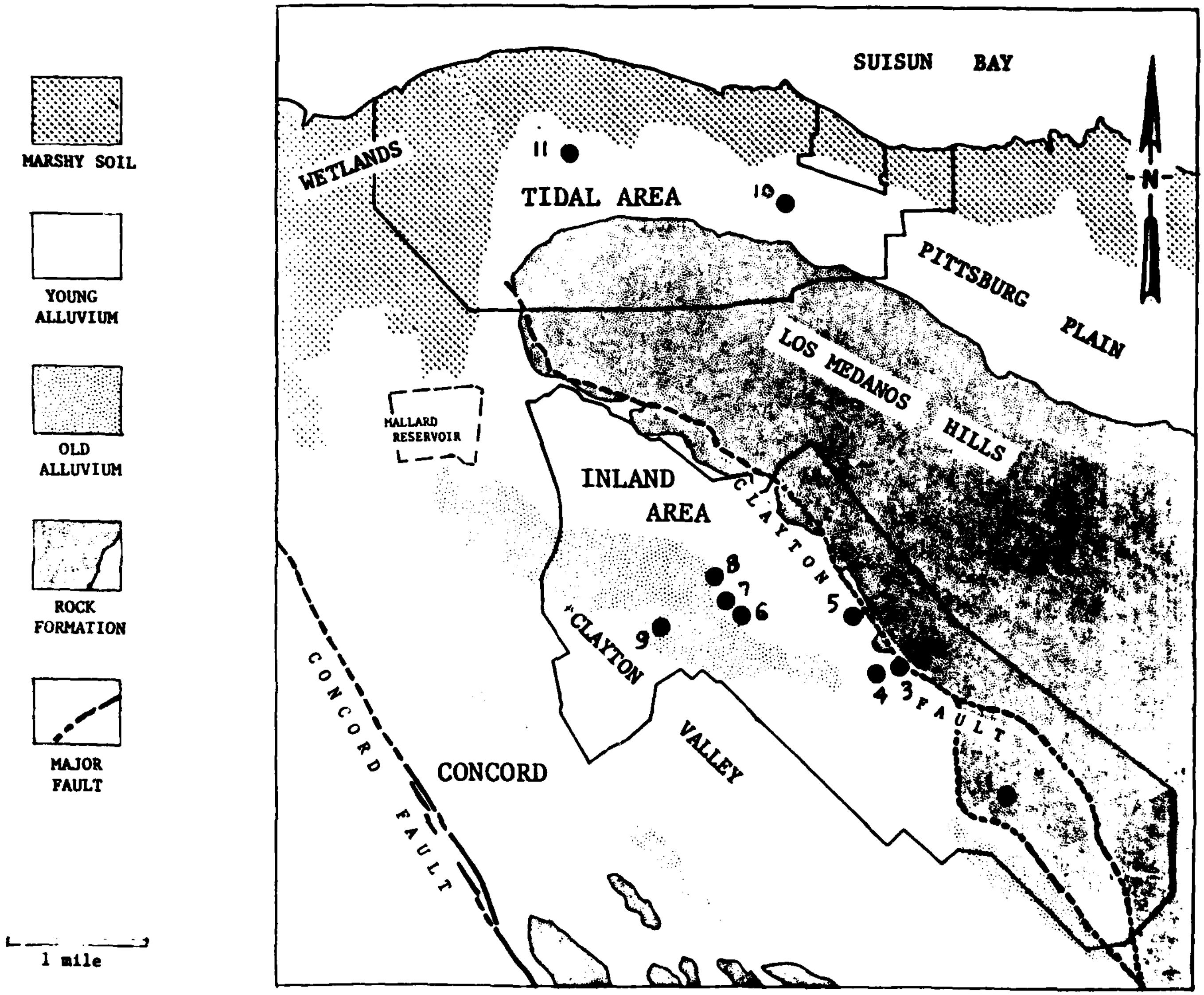

Figure 1. Locations of eleven sites on Concord Naval Weapons Station

soils. Silt sizes probably predominate. A 3-ft* thick layer of dark brown or gray, clayey residual soil is consistently present on the alluvium throughout the region. This clay has expansive characteristics and is locally called gumbo.

8. Water wells have been abundant in all lowland areas where alluvium containing sand and gravel beds are found. Water was raised for agricultural and domestic consumption in the past but now the water goes mostly to industrial users at a smaller level of demand. A well field around Mallard Reservoir (Figure 1) penetrates interbedded sand and fine-grained layers to as deep as 500 ft. The water table is related to the level of Suisun Bay near that water body. Away from the bay (e.g. up Clayton Valley) the water table seems to parallel roughly the surface topography at a depth up to 50 ft.

9. Figure 1 also shows the two major faults known to cross the region. The Concord fault passes through the city of Concord at a distance of approximately 2 miles from the NWS Concord. The fault is classified as active by the California Division of Mines and Geology as evident by offset of curbs and other man-made features (Sharp 1973). The Clayton fault bounds Los Medanos Hills in passing through NWS Concord. This fault has been classified as inactive.

References

10. Selected references useful in synthesizing local geology and related background are listed under References at the end of this report. Four preliminary geologic quadrangle maps were directly helpful in the preliminary screening of sites (Dibblee 1980a, 1980b, 1980c, and 1981). Other useful references are Helley et al. (1979), Thomasson et al. (1960), and Wesnousky (1986).

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* A table of factors for converting Non-SI units of measurement to SI (metric) units is presented on page 4.

PART II: SUITABILITY CRITERIA

11. Federal statutes and regulations with which the Navy may have to comply in siting a Class I RCRA-type facility on NWS Concord include:

a. The Resource Conservation and Recovery Act, as amended, 42 U.S.C. 6901 et. seq.

b. Standards for Owners and Operators of Hazardous Waste Treatment, Storage, and Disposal Facilities, 40 C.F.R. 264.

c. EPA Administered Permit Programs: The Hazardous Waste Permit Program, 40 C.F.R. 270.

12. State statutes, regulations, and instructions with which the Navy may have to comply in siting a Class I RCRA-type facility on NWS Concord include:

a. California Hazardous Waste Control Act, California Health and Safety Code, Division 20, Chapter 6.5, Article 9. Permitting of Facilities.

b. California Hazardous Waste Management Regulations, California Administrative Code, Title 22. Social Security, Division 4, Chapter 30. Minimum Standards for Management of Hazardous and Extremely Hazardous Wastes, Article 4. Hazardous Waste Facility Permit.

c. California Porter-Cologne Water Quality Act, California Water Code, Division 7. Water Quality.

d. California Water Regulations, California Administrative Code, Title 23. Waters, Chapter 3, Subchapter 15. Discharges of Waste to Land.

e. Instructions for Preparing a Part B Application for a Hazardous Waste Disposal Facility, Department of Health Services, Toxic Substances Control Division, June 1985.

13. Pertinent criteria concern the suitability of a site with regard to threats of flooding, earthquakes, settlement, and landsliding; potential for pollution of ground water; and creation of a buffer zone.

Flooding Potential

14. The California Water Regulations prohibit the siting of Class I waste management units for hazardous waste within the 100-year floodplain unless such units are designed, constructed, operated, and maintained to

prevent inundation or washout because of floods within a 100-year return period. The California Water Regulations provide that:

> Flooding--New disposal units and existing units in Category I other than existing land treatment units, shall be located outside of floodplains subject to inundation by floods with a 100-year return period.

[CAL. ADMIN. CODE, Section 2531(c)]. The instructions for preparing a Part B application also stress the advantages of locations above the 100-year flood level as follows:

> Owners and operators of all facilities shall provide an identification of whether the facility is located within a 100-year floodplain. This identification shall indicate the source of data for the determination and include a copy of the relevant Federal Insurance Administration (FIA) flood map, if used, or the calculations and maps used where a FIA map is not available. Information shall also be provided identifying the 100-year flood level and any other special flooding factors (e.g., wave action) which must be considered in designing, constructing, operating, or maintaining the facility to withstand washout from a 100-year flood.

[Instructions for Preparing a Part B Application for a Hazardous Waste Facility, General Information Requirements, page A-52]. Locations below the 100-year flood level require engineered features resisting erosion or provisions for relocating waste whenever a flood is probable. Either of these contingencies will increase cost substantially and on that basis should be avoided. Accordingly, location above the 100-year flood is favored.

Earthquake Potential

15. The California Water Regulations provide in regard to earthquake potential that:

> Ground Rupture--New units and existing units in Categories I, I', REC, and EX, other than existing land treatment units, shall have a 200-foot setback from any known Holocene fault.

[CAL. ADMIN. CODE, Section 2531(d)]. The instructions for preparing a Part B application require that:

> No faults which have had displacement in Holocene time are present, or no lineations which suggest the presence of a fault (which have displacement in Holocene time)

within 3,000 feet of a facility are present, based on data from:

- o Published geologic studies;
- o Aerial reconnaissance of the area with a five-mile radius from the facility;
- o An analysis of aerial photographs covering a 3,000-foot radius of the facility; and
- o If needed to clarify the above data, a reconnaissance based on walking portions of the area within 3,000 feet of the facility; or

If faults (to include lineations) which have had displacement in Holocene time are present within 3,000 feet of a facility, no faults pass within 200 feet of the portions of the facility where treatment, storage, or disposal of hazardous waste will be conducted, based on data from a comprehensive geologic analysis of the site. Unless a site analysis is otherwise conclusive concerning the absence of faults within 200 feet of such portions of the facility, data shall be obtained from a subsurface exploration (trenching) of the area within a distance no less than 200 feet from portions of the facility where treatment, storage, or disposal of hazardous waste will be conducted. The trenching shall be performed in a direction that is perpendicular to known faults (which have had displacement in Holocene time) passing within 3,000 feet of the portions of the facility where treatment, storage, or disposal of hazardous waste will be conducted. The investigation shall document, with supporting maps and other analyses, the location of any faults found.

[Instructions for Preparing a Part B Application for a Hazardous Waste Facility, General Information Requirements, pages A-51 and A-52].

Landsliding Potential

16. The California Water Regulations prohibit the siting of Class I waste management units for hazardous waste within areas where landsliding potential exists. Specifically, those regulations provide that:

> Rapid Geologic Change-New disposal units and existing units in Categories I, I', and EX, other than existing land treatment units, shall be located outside areas of potential rapid geologic change.

[CAL. ADMIN. CODE, Section 2531(e)].

Wave Potential

17. The California Water Regulations prohibit the siting of Class I waste management units for hazardous waste within areas where wave potential exists. Specifically, those regulations provide that:

> Tidal Waves--New disposal units shall be located outside areas subject to tsunamis, seiches, and surges.

[CAL. ADMIN. CODE, Section 2531(f)]. This siting criterion would be applicable only to the Tidal Area of NWS Concord (Figure 1).

Settlement Potential

18. The California Water Regulations prohibit the siting of Class I waste management units for hazardous waste within areas where settlement potential exists. Specifically, the regulations provide that:

> Rapid Geologic Change--New disposal units and existing units in Categories I, I', and EX, other than existing land treatment units, shall be located outside areas of potential rapid geologic change.

[CAL. ADMIN. CODE, Section 2531(e)]. Damaging settlement is prohibited by the California Water Regulations as follows:

> All containment structures at waste management units shall have a foundation or base capable of providing support for the structures and capable of withstanding hydraulic pressure gradients to prevent failure due to settlement, compression, or uplift as certified by a registered civil engineer or certified engineering geologist.

[CAL. ADMIN. CODE, Section 2530(d)].

Liquefaction Potential

19. Liquefaction qualifies as a process of rapid geological change so that the California Water Regulations prohibit the siting of Class I waste management units for hazardous waste within areas of liquefaction potential. Specifically, the California Water Regulations provide that:

> Rapid Geologic Change--New disposal units and existing units in Categories I, I', and EX, other than existing

> land treatment units, shall be located outside areas of potential rapid geologic change.

[CAL. ADMIN. CODE, Section 2531(e)]. In addition, the California Water Regulations require that:

> All containment structures at waste management units shall have a foundation or base capable of providing support for the structures and capable of withstanding hydraulic pressure gradients to prevent failure due to settlement, compression, or uplift as certified by a registered civil engineer or certified engineering geologist.

[CAL. ADMIN. CODE, Section 2530(d)].

Ground-Water Protection

20. The California Water Regulations provide that:

> All new landfills, waste piles, and surface impoundments shall be sited, designed, constructed, and operated to ensure that wastes will be a minimum of 5 feet above the highest anticipated elevation of underlying ground water.

[CAL. ADMIN. CODE, Section 2530(c)]. More specifically, the California Water Regulations provide that:

> Class I disposal units shall be located where natural geologic features provide optimum conditions for isolation of wastes from waters of the state.

In regard to the geological setting for Class I wastes,

> New and existing Class I units shall be immediately underlain by natural geologic materials which have a permeability of not more than $1x10^{-7}$ cm/sec, and which are of sufficient thickness to prevent vertical movement of fluid, including waste and leachate, from waste management units to waters of the state as long as wastes in such units pose a threat to water quality. Class I units shall not be located where areas of primary (porous) or secondary (rock opening) permeability greater than $1x10^{-7}$ cm/sec could impair the competence of natural geologic materials to act as a barrier to vertical fluid movement. These provisions do not apply to land treatment facilities.
>
> Natural or artificial barriers shall be used to prevent lateral movement of fluid, including waste and leachate.

[CAL. ADMIN. CODE, Sections 2531(a) and (b)(1)]. The instructions for preparing a Part B application provide that:

> The owners and operators of surface impoundments, waste piles, land treatment facilities, and landfills shall provide information regarding the depth to the saturated zone or ground-water table, including seasonal high levels for ground water, known aquifers beneath the site, and any aquifers having hydraulic continuity.

[Instructions for Preparing a Part B Application for a Hazardous Waste Facility, General Information Requirements, page A-53].

Buffer Zone

21. The California Hazardous Waste Control Act sets forth the requirements for a buffer zone around Class I waste management units for hazardous wastes:

> Except as provided ...[below]... any land on which is located a hazardous waste disposal facility permitted pursuant to this chapter shall be surrounded by a minimum buffer zone of 2,000 feet between the facility and the outer boundary of the buffer zone. The department may impose an easement, covenant, restriction, or servitude, or any combination thereof, as appropriate, on the buffer zone.... If the department determines that a buffer zone of more than 2,000 feet is necessary to protect the present and future public health and safety, the department may increase the buffer zone by restricting the disposal of hazardous waste at that facility to land surrounded by a larger buffer zone.
>
> If the owner of a hazardous waste disposal facility proves to the satisfaction of the department that a buffer zone of less than 2,000 feet is sufficient to protect the present and future public health and safety, the department may allow the disposal of hazardous waste onto land surrounded by a buffer zone of less than 2,000 feet.

[CAL. HEALTH AND SAFETY CODE, Section 25202.5].

PART III: SITE SCREENING

22. All eleven sites (Figure 1) identified as potential sites were screened for suitability for Class I waste disposal on the basis of existing information. The criteria under which the sites were evaluated were those presented in Part II. The eleven sites constituted all the available land on the NWS Concord and were mostly more than 2,000 ft from the boundary. Sites with this standoff from private property satisfy the State of California requirement for a buffer zone. Exceptions are Sites 2 and 10 with special circumstances. Each site is reviewed below for preliminary suitability and to explain whether subsurface investigations were considered warranted.

Site 1

23. Site 1 is located immediately north of the intersection of Kinne Blvd. with Bailey Road in Section 34, T2N, R1W (Figure 2). The area covers approximately 58 acres. The northern half of the site occupies a broad ridge underlain by sandstone and dacite. Although exposures in railroad cuts suggest that much of the dacitic material is rubbly and fragmented, one dike was recognized as intruding sandstone.

24. The south half of Site 1 occupies low land along a short, unnamed tributary to Mount Diablo Creek. Because a branch of the Clayton fault apparently passes south of Site 1 (Figure 1), it is reasonable to presume that the rocky material found on the broad ridge continues into the low land beneath a veneer of Holocene alluvium.

25. Site 1 is unsuitable for a hazardous waste landfill primarily because the jointed sandstone and dacite and rubbly rock deposits observed at the site will have a high secondary permeability and will not meet the State criterion of low permeability. Subsurface testing was considered to be unnecessary to prove this deficiency, and Site 1 was dropped from further consideration.

Site 2

26. Site 2 occupies steep terrain along the southern flank of Los Medanos Hills near the NWS Concord pistol range in Section 28, T2N, R1W

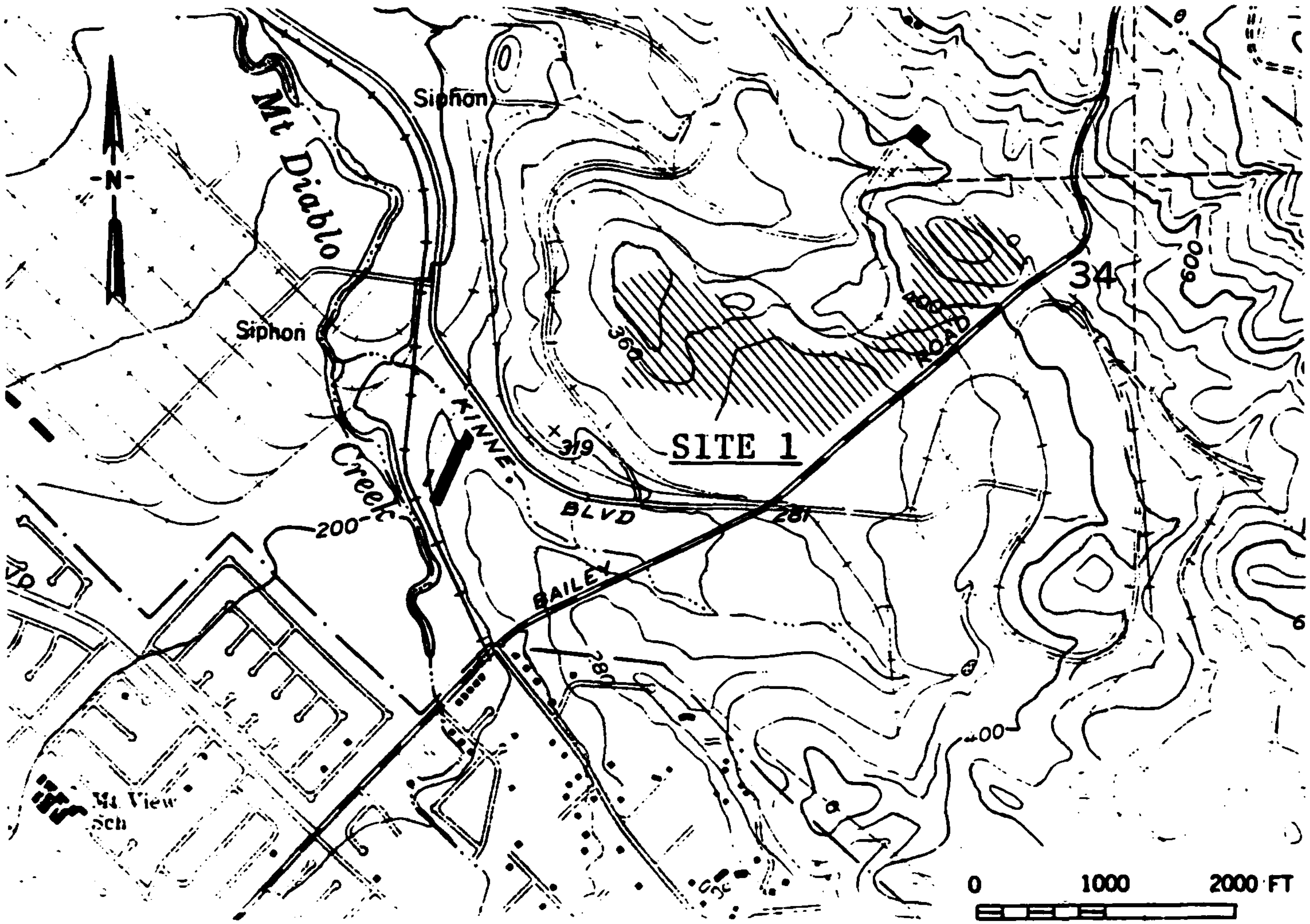

Figure 2. Site 1 and surroundings

(Figure 3). The area covers approximately 34 acres. That portion within 2,000 ft of the NWS Concord boundary is still remote from dwellings and thus sufficiently buffered. Steeply dipping sandstone beds underlying the site are of Tertiary age. Slope creep is locally evident, and a few landslides have been identified from aerial photographs. A landslide formed 5,000 ft away at Building 87 on Mott Drive but was largely restricted to fill material associated with construction. That slide nevertheless is indicative of one problem to be expected in working in this steep terrain.

27. Site 2 is unsuitable for a hazardous waste disposal site primarily because the jointed nature of sandstone produces secondary permeability that can be expected to exceed the State's low-permeability criterion. The other major shortcoming of Site 2 is the steep terrain which will require extensive cut and fill and will be conducive to landsliding. The site was dropped from further consideration.

Site 3

28. Site 3 lies in Section 28, T2N, R1W at the north side of Clayton Valley and adjacent to Los Medanos Hills (Figure 3). The area covers approximately 14 acres. The boundary between the Tertiary rock formations of the hills and the Holocene alluvium of the valley is the Clayton fault. The fault is classified by the California Division of Mines and Geology as inactive. The necessary 200-ft setback from the Clayton fault reduces the relatively small area of the site even further.

29. The main disadvantage of Site 3 is the presence of landslides as outlined by Earth Sciences Associates (1982) from aerial photographs. Although some of this previously recognized disturbance appears on close inspection to result from excavation and fill activity, the likelihood of past disturbance of the alluvium and the potential for instability continuing into the future remain as serious shortcomings. Accordingly, Site 3 was dropped from further consideration.

Site 4

30. Site 4 is located on the north side of Claytcn Valley upslope from Building 1A-24 in Section 28, T2N, R1W (Figure 3). The area covers

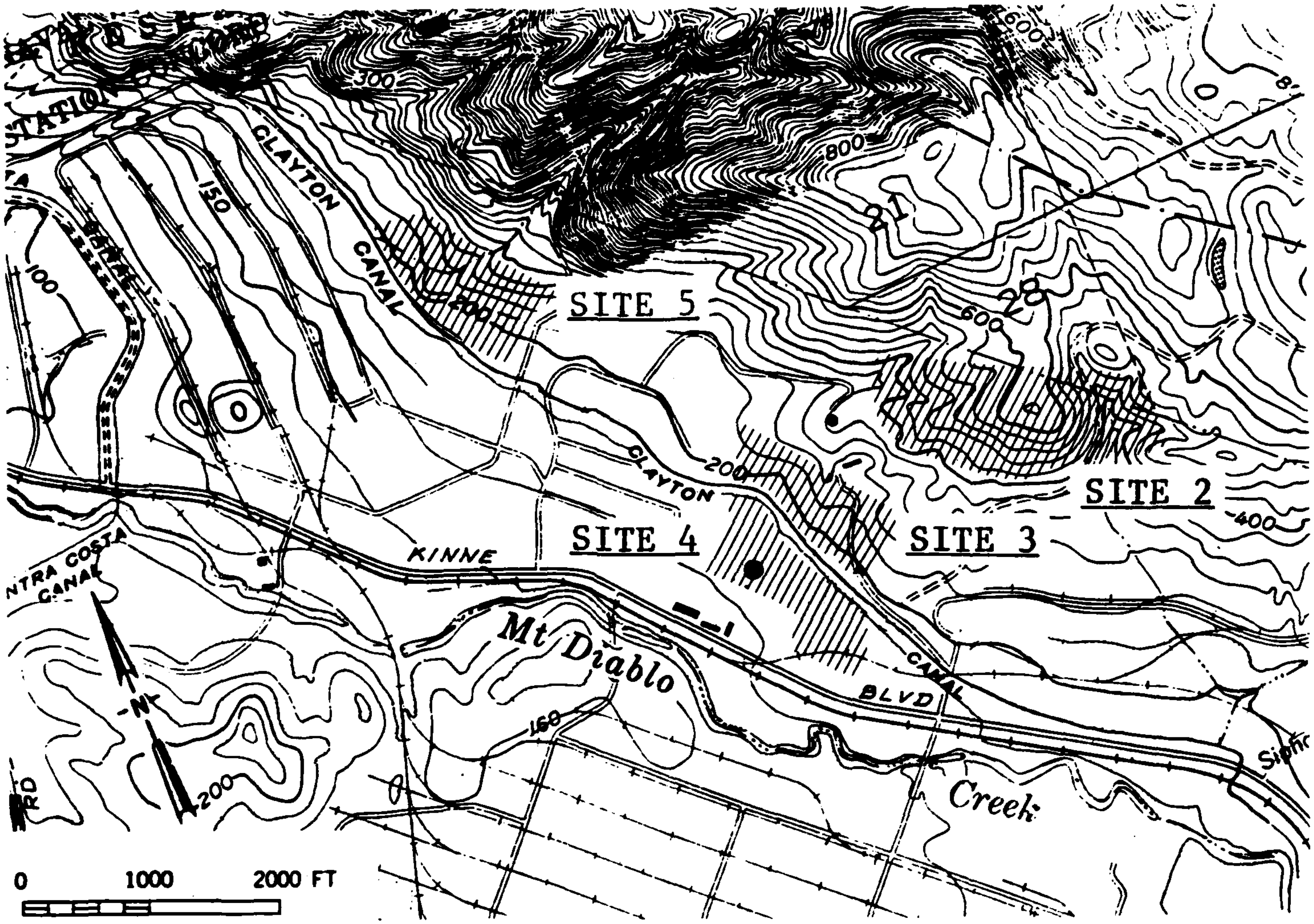

Figure 3. Sites 2, 3, 4, and 5 and location of drill hole (solid circle)

approximately 15 acres. The site appeared to satisfy criteria other than those concerned with protection of ground water and therefore was selected for subsurface investigation. The evaluation of Site 4 is presented in Part VI.

Site 5

31. Site 5 is located in Section 21, T2N, R1W on the north side of Clayton Valley (Figure 3) upslope from Magazines 2AT76 and 2AT77. The area covers approximately 17 acres. The terrain has the configuration of an alluvial fan accumulated southwest of the mouth of a canyon in Los Medanos Hills. The alluvium was expected to be clay-rich like the Tertiary formations in the hills which made it promising as a site for hazardous waste landfilling.

32. In December 1986 water was found locally at and near the ground surface along the intermittent runoff channel from the source canyon. This channel crosses the center of Site 5 indicating that perched water rises to the surface at times during the year in violation of one of the State criteria against ground-water pollution. Because of this serious defect, Site 5 was rejected as undeserving of subsurface investigation and further consideration.

Sites 6, 7, and 8

33. Sites 6, 7, and 8 are 13-, 15-, and 26-acre areas, respectively, situated in Clayton Valley southwest of the floodplain of Mount Diablo Creek (Figure 4). In this position the soils at the three sites are expected to consist of a similar combination of Holocene alluvium related to Mount Diablo Creek over stiffer, older alluvium forming the broad hills further southwest. The Holocene alluvium presumably thins out completely near the southwest edge of the floodplain. The boundary between younger and older alluvium evidently passes through the sites but could not be pinpointed in the field.

34. Sites 6, 7, and 8 appeared to satisfy the criteria other than those concerned with protection of ground water. Two were investigated by drilling and sampling. Evaluations of Sites 7 and 8 are presented in Part VI. Site 6 was not drilled, but its subsurface is considered to be probably similar to that at Site 7.

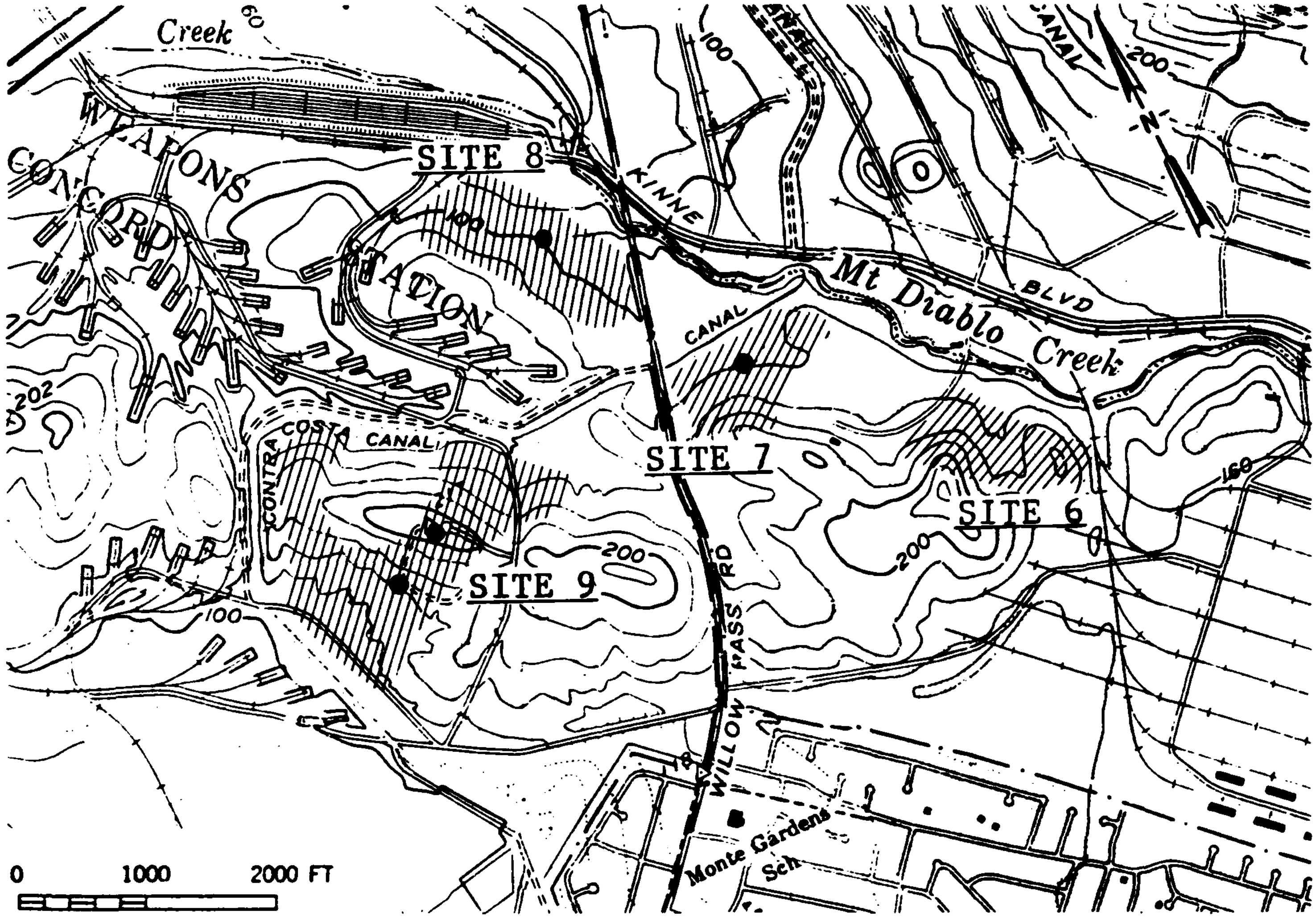

Figure 4. Sites 6, 7, 8, and 9 and locations of drill holes (solid circles)

Site 9

35. Site 9 is located in Clayton Valley on the southwestern flank of the band of broad hills crossing Willow Pass Road southwest of Mount Diablo Creek (Figure 4). The area is approximately 40 acres. Site 9 appeared to satisfy the criteria other than those concerned with protection of ground water. Therefore it was selected for subsurface investigation. The evaluation of Site 9 is presented in Part VI.

Site 10

36. Site 10 lies northwest of the intersection of Nichols Road and Port Chicago Highway in Section 5, T2N, R1W (Figure 5). The area is approximately 36 acres. The site is on a large alluvial fan located on the Pittsburg Plain and downslope from the mouth of a canyon from Los Medanos Hills. The narrow buffer of less than 1,000 ft along the east side is unfavorable and will need to be weighed in future evaluations against favorable features, location, and subsurface conditions as discussed in Part VI.

Site 11

37. Site 11 is located 0.3 miles north of the Coast Guard station north of Port Chicago Highway in Section 6, T2N, R1W (Figure 6). The area is approximately 40 acres. The site is narrow but extends about 1 mile east-west just south of the railroad right-of-way. The site is at the lower edge of the Pittsburg Plain. Ground surface is within about 10 ft of the level of Suisun Bay, and the presence of marshy ponds confirms a high-standing water table. The high water table makes this site unsuitable, and thus the site was not given further consideration.

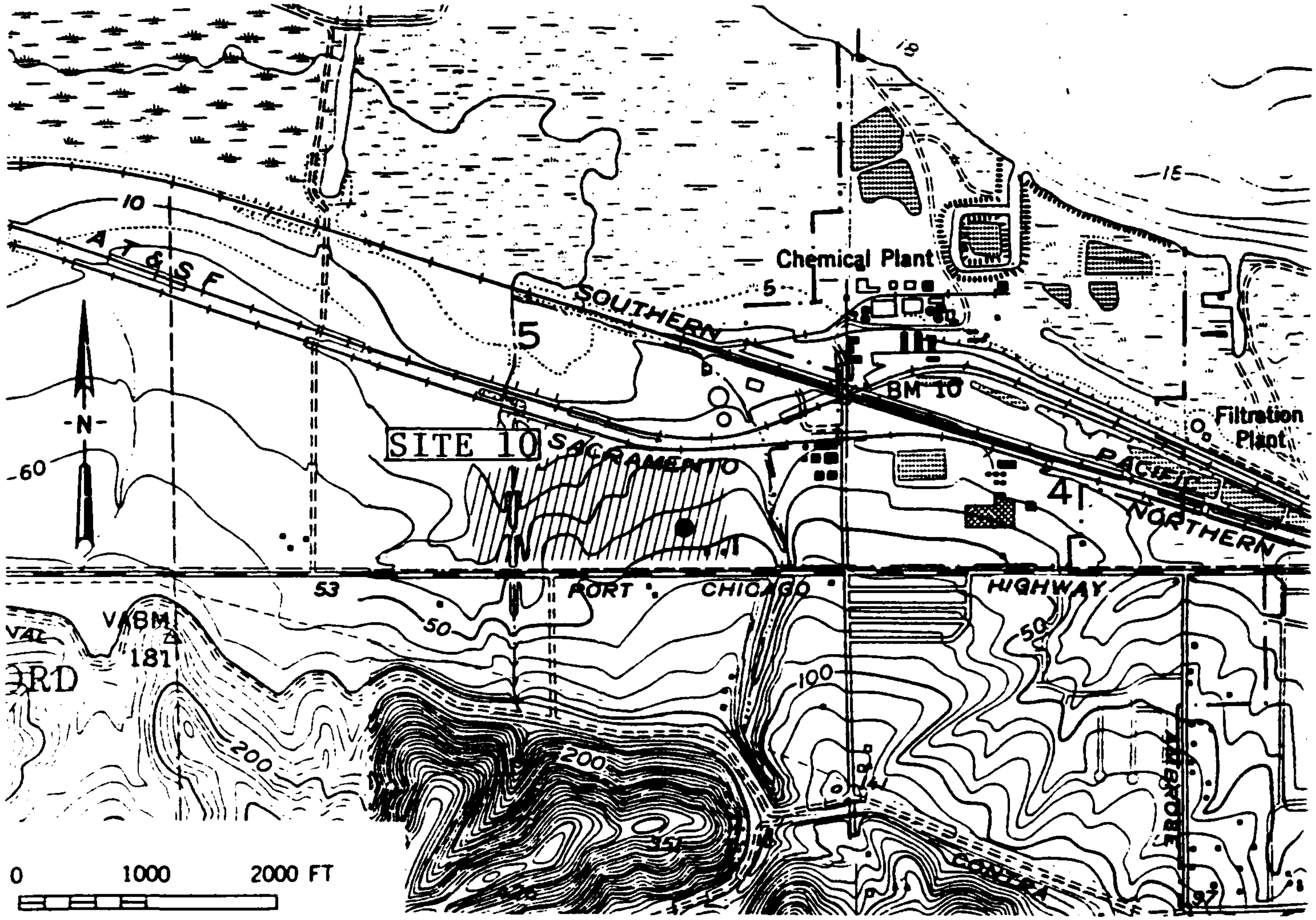

Figure 5. Site 10 and location of drill hole (solid circle)

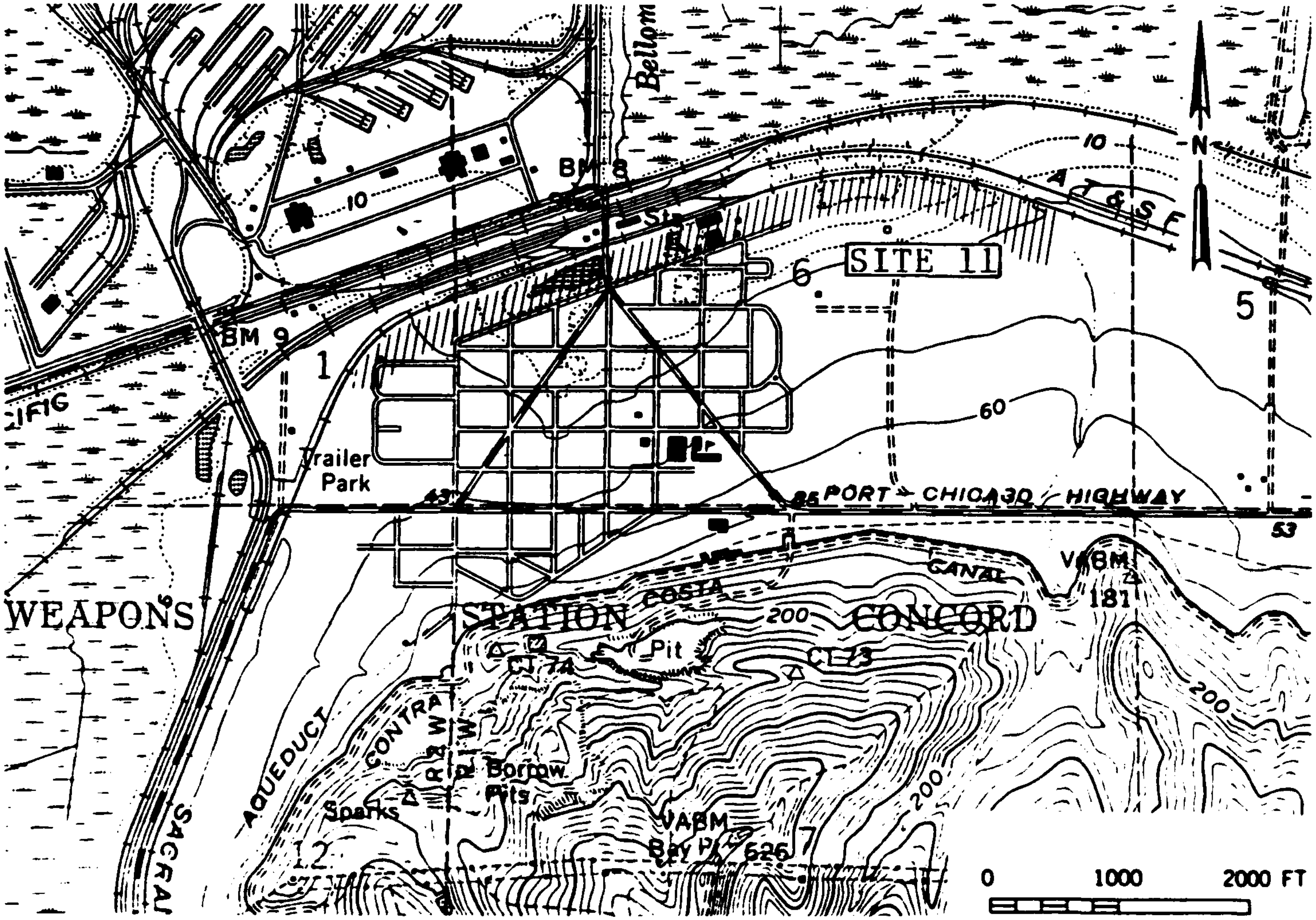

Figure 6. Site 11 and surroundings

PART IV: FIELD PROCEDURES

38. The procedures and equipment used in investigating five sites surviving the site screening, Sites 4, 7, 8, 9, and 10, are presented below. All sites were located in alluvial settings. At each site the holes for subsurface investigation were placed close together (about 10 to 25 ft) to facilitate correlation of data and results. This clustering limits the subsurface investigations to a one-dimensional picture.

Drilling

39. All drilling was accomplished with a Failing 1500 truck-mounted rig having a capability well beyond the depths explored in this investigation. For the first site, three holes were drilled: the first to explore strata to just below the level expected for the water table, the second to recover soil samples and develop a continuous geological log, and the third to conduct field permeability tests. On subsequent sites the unnecessary exploratory hole was eliminated to reduce cost. Then only two holes were drilled to sample and test each site, except where early logging indicated unsuitable materials. Individual sites are reviewed in Part VI.

Sampling holes

40. A 6-3/4-in. diam sampling hole was drilled to a level at least 10 ft below that where ground water was encountered. Soils above the water table are somewhat dry and can be distinguished from saturated soil below the water table. Undisturbed core 5 in. in diameter was recovered continuously using a 5 by 6-1/4-in. double tube core barrel with Shelby tube or by pushing with a Hvorslev sampler, depending on soil characteristics. Soil was forcefully extruded from the steel sampling tube immediately upon recovery to minimize drying and sticking to the cylinder wall. Samples for testing in the laboratory were selected from the soil cores as recovered and were preserved as described in paragraph 47.

41. Core not preserved for testing was cleaned with water or was cut lengthwise for detailed examination of a fresh surface. Otherwise, the clay released from the natural strata in the process of drilling mixed with the drilling water and produced mud that coated the core and obscured natural stratification. Pebbles dispersed through some silty strata tended to

accumulate in the hole during drilling and flushing and was recovered at the top of each core run. Also, the pebbles were commonly stuck in the muddy coating on core as recovered. Both of these deceptive pebble accumulations were recognized early and seldom if ever mistaken for natural gravel or gravelly layers.

42. Pebbly strata presented problems for the drilling operation, and only a few representative samples were recovered. The leading edge of the sampling cylinder was bent on several instances as pebbles were encountered, and more than once a tube was jammed and deformed in passing over stones that accumulated in the bottom of the hole. Generally, the results were good in soil with no more than dispersed pebbles.

Testing holes

43. The drilling of the holes for permeability field testing was an integral part of the field testing. Details are presented in paragraph 48.

Piezometer holes

44. Holes drilled for sampling or for field testing were used for installing piezometers. Details are presented in paragraphs 50 through 53.

Documenting

45. Geological details were recorded during close examination of the core immediately after extrusion from the sampling tube. Core pieces were retained in wooden troughs until completion of drilling for comparison of portions of the full section. Generally, the first classification and descriptions remained valid. Cracking that followed exposure and drying was of some value in delineating clay-rich intervals. The few relatively clean sand layers were also easily distinguished as they dried and crumbled in handling.

46. The logging of core was carefully integrated with selections of samples for laboratory testing and intervals for field testing in order to have test results representing the major soil classes and strata. The logs presented in Part VI for sampling holes include all pertinent geotechnical data and interpretations from the field.

Sampling and Preserving

47. Each laboratory undisturbed sample was carefully preserved by wrapping first in parafilm and then aluminum foil. The seam in the foil was taped, and the sample was transferred to a 6-in. diam waxed cardboard tube. Melted wax was poured over the sample to encase completely the sample and minimize the likelihood of moisture change or disturbance prior to testing. Samples were transported to the testing laboratory within 2 weeks. The samples were stored at the laboratory in a room with humidity and temperature controlled to minimize change before and during preparations for testing.

Field Permeability Testing

48. Field permeability testing was conducted using a falling-head slug test modified for the unsaturated zone. A convenient short review of field tests is found in Olson and Daniel (1981). Test intervals were selected using logs from sampling holes located no more than 20 ft away. The procedures are described here and results are presented in Part VI. A period of soaking to approach saturation preceded the usual steps. The following steps formulated for the interval 10.8 to 15.3 ft at Site 10 were used generally thereafter on the other sites.

a. Drill to 5 ft using NX rotary tricone bit and water. Insert 5-ft section of 3-in. ID casing into hole with 3-1/2-in. drag bit attached to bottom. Rotate casing and bit using water to extend hole to 9.5 ft, then push casing 1.3 ft to a depth of 10.8 ft to achieve a good seal against the wall. (The drag bit was omitted in later testing.)

b. Insert drill rod and NX bit through casing and drag bit and extend hole to 15.3 ft using clean water to remove cuttings. Drilling fluid was not recirculated but rather was wasted to avoid mudding the wall of the test section. Upon reaching the bottom depth of the test section, circulate clean water through the hole until return water clears and continue for an additional 5 min.

c. Fill hole with clean water to the top of the casing and allow to stand to achieve saturation of the test section. The soaking period lasts 1 to 2 hr.

d. Begin the test by refilling the casing. Take readings at convenient head-drop increments or time intervals.

An M-scope is used to sense the water level, and a watch is used for the time.

This general procedure was usually followed on subsequent testing, but a notable exception was necessary where soil was susceptible to caving. The procedure was modified as follows:

a. Extend hole to depth corresponding to bottom of test section by pushing a 3-in. Shelby tube and recovering core repeatedly to the desired depth. The use of water is avoided to prevent hydrofracturing of soil strata.

b. Push 3-in. ID by 3-1/2-in. OD steel casing to depth corresponding to top of test section.

c. Insert small drill rod through casing to bottom of hole to clear any plugs that may have occurred while pushing the casing.

d. Insert 2-in. ID PVC well screen of same length as test section with solid PVC riser casing sections attached to bring the string to about the same level above ground surface as steel casing.

e. Fill hole with water. Both the PVC riser and the 3-in. ID steel casing are filled with clean water. No seal exists between the PVC riser and steel casing, and the water level falls at the same rate in both during the test. Some uncertainty existed about the effective diameter of the test section. If the hole did not cave, the diameter is the original 3 in. Partial caving was considered to dictate some value between 2.5 and 3 in. The error that could result from choosing one or the other diameter was neglected in view of the imprecision of other test variables.

f. Allow soil surrounding the test section to saturate and conduct test in the manner described previously.

49. The coefficient of permeability was calculated primarily by using the following equation:

$$k = \frac{A}{FDt} \ln \frac{h_1}{h_2}$$

where

A = cross-sectional area of riser

F = shape factor, related to geometry of test section

D = diameter of test section

h_1 = head at time t_1

h_2 = head at time t_2

t = time difference t_2-t_1

and where data were available, the secondary equation from Hvorslev (1951) was also used

$$k = \frac{d^2 \ln\left(\frac{2L}{D}\right)}{8LT}$$

where

d = diameter of riser or casing

L = length of test section

T = time lag factor, i.e. the elapsed time corresponding to 37 percent of the original head remaining as defined by Hvorslev.

The shape factor for tests at NWS Concord was calculated using Hvorslev's equation for an open test section of given length and diameter in a uniform soil below a cased section.

$$F = \frac{2\pi(L/D)}{\ln\left[L/D + \sqrt{1 + (L/D)^2}\right]}$$

Table 1 contains the results of falling head tests conducted in the field at Sites 4, 7, 9, and 10.

Piezometer Installation

50. Two types of piezometers were installed to locate the water table accurately and to record its fluctuation. A 2-in. screen and riser combination was used where there was a possible need to obtain a sample of water (e.g., for background water quality). This possible need has not yet been established. For the water level only, a 3/4-in. stock piezometer tip and riser combination was used. The actual installations and early observations are described in Part VI.

51. The 2-in. monitoring wells were installed at Sites 7 and 10 with 5-ft PVC screened interval and 0.5-ft bottom stub. The screened section, end cap, and PVC riser were assembled, lowered into the 3-in. ID casing, and held in place while sand was poured into the annulus between the 2-in. riser and the 3-in. casing. When sand had been placed to about 10 ft above the screened interval, the casing of the boring was extracted and bentonite pellets were

Table 1

Permeability from Field Tests in Drill Holes

Depth, ft From	Depth, ft To	Coefficient of Permeability, cm/sec	Remarks
		Site 4	
5.0	13.0	2×10^{-6}	Fair
13.0	17.0	3×10^{-6}	Good
18.4	21.1	2×10^{-4}	Poor; no straight line (2 tests).
28.8	32.4	5×10^{-3}	Rough; but probably satisfactory.
32.5	41.0	8×10^{-7}	Good
37.3	52.0	1×10^{-3}	Good; 2 tests.
		Site 7	
6.5	11.6	2×10^{-6}	Good
12.0	15.3	1×10^{-6}	Good
14.1	19.0	4×10^{-6}	Good
14.1	25.1	1×10^{-6}	Good
24.0	37.5	7×10^{-7}	Good
34.5	45.0	6×10^{-6}	Good; test run in piezometer.
		Site 9	
7.6	17.6	6×10^{-6}	Fair; caved during test.
17.9	23.5	2×10^{-6}	Good
25.0	35.0	6×10^{-5}	Fair; unsure of seal.
30.0	38.0	2×10^{-5}	Fair; unsure of seal.
40.7	45.3	4×10^{-6}	Good
45.5	50.9	3×10^{-6}	Good
52.0	69.0	4×10^{-6}	Good
		Site 10	
10.8	15.3	2×10^{-5}	Good
26.0	36.0	2×10^{-6}	Good
35.1	45.1	1×10^{-5}	Good
45.0	55.5	3×10^{-5}	Poor; caved during test.

placed in the annulus to seal the hole. Bentonite pellets were placed to a depth of about 2 ft above the sand; then cement was placed to 6 ft below the ground surface. The 2-in. riser extended to about 1.5 ft above the ground surface. The hole was backflushed with air or water to expel fines that had entered the screened interval. A protective steel sleeve and lockable cap were fitted over each riser and grouted in place. Painted wooden posts were placed around the sleeve for protection.

52. Piezometer tips and risers measuring 3/4 in. in diameter were used at Sites 4 and 9. The basic installation procedure was the same as was used for the 2-in. diam wells.

53. Table 2 shows the results of measurements in piezometers. Most water levels stabilized soon after installation, and little or no fluctuation in level is evident during the short period of measurements. The apparent discontinuity in readings at Site 10 late in February probably reflected a previous error in not subtracting casing or riser stickup.

Table 2

Water Levels in Piezometers

Site	Depth to Tip or Screen, ft	Date*	Depth from Surface, ft
4	46	3-17	42.6
		3-23	42.2
		3-25	42.4
		4-02	42.7
7	40-45	3-23	34.5
		3-25	34.4
		3-26	34.3
		4-02	34.4
9	105	2-24	48.2
		2-25	48.0
		2-26	48.1
		2-27	48.2
		3-06	48.6
		3-16	48.5
		3-21	48.4
		3-25	48.4
		4-02	48.3
9	50	3-06	45.3
		3-16	47.1
		3-21	47.3
		3-25	47.3
		4-02	47.3
10	(open hole)	2-12	41.6
	(open hole)	2-13	41.5
	49-54	2-19	41.6
		2-28	39.1
		3-21	38.9
		4-02	39.1
		5-12	39.5

* In 1987.

PART V: LABORATORY TESTING PROCEDURES

54. Laboratory testing was undertaken to characterize the soils generally and to quantify permeability specifically. Methods are addressed below and test results are tabulated. Interpretations of results are presented in Part VI.

Test Selection

55. Appropriate index properties are useful in comparing materials from different layers. Atterberg limits are particularly suited to the NWS Concord study with its emphasis on clay-bearing soils since these indices bracket the plasticity range of fine-grained soils. Grain-size distribution is a second characteristic important not only for indexing a soil but also for revealing subtle characteristics affecting soil permeability. Both mechanical and hydrometer analyses were made for grain sizes. The dry density or dry unit weight is determined incidentally to calculating the coefficient of permeability k. Dry density is useful because it is an indicator of pore space and soil stiffness and ocasionally even the relative age of soil.

56. Two tests were selected to measure soil permeability. One test used a triaxial cell capable of relatively high pressure, and the other confined the soil in a pressure chamber at only moderate pressure. The triaxial cell was used with fine-grained soils expected to have k values less than 10^{-4} cm/sec since backpressuring is necessary to reduce the impeding effects of air bubbles.

Atterberg Limits

57. The standardized testing procedures that were followed for plastic limit and liquid limit were:

- a. "Liquid and Plastic Limits," Appendix III, Laboratory Soils Testing, Engineer Manual EM 1110-2-1906, US Army Corps of Engineers, 1970, (with change 2 dated 1986), pp III-1 to III-31.
- b. "One-Point Liquid Limit Test," Appendix IIIA, Laboratory Soils Testing, Engineer Manual EM 1110-2-1906, US Army Corps of Engineers, 1970, (with change 2 dated 1986), pp IIIA-1 to IIIA-3.

Similar methods are described in the ASTM standards as follows:

ASTM D4318-84, "Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils," Annual Book of ASTM Standards: Volume 04.08 Soil and Rock; Building Stones.

The results of testing are shown in Table 3.

Grain-Size Distribution

58. Grain-size distribution is determined by sieve analysis and hydrometer analysis for materials retained on and passing the US Standard No. 200 sieve, respectively. The sieve analysis consists of passing a sample through a set of sieves and weighing the amount retained on each. The analysis is generally performed on material retained on a No. 200 sieve.

59. The hydrometer analysis is conducted on the material passing the No. 200 sieve to determine the percentage of dispersed soil particles remaining in suspension at a given time, with finer particles in suspension longer. Standard procedures are presented in:

a. "Grain-Size Analysis," Appendix V, Laboratory Soils Testing, Engineer Manual EM 1110-2-1906, US Army Corps of Engineers, 1970, (with change 2 dated 1986), pp V-1 to V-28.

b. ASTM D422-63 (Reapproved 1972), "Standard Method for Particle-Size Analysis of Soils," Annual Book of ASTM Standards: Volume 04.08 Soil and Rock; Building Stones, 11 pp.

The results of sieve analyses are presented in Table 3. Results of hydrometer analyses are shown in Figures 7 and 8.

Dry Unit Weight

60. The method for determining dry unit weight is presented along with methods for determining other basic soil parameters in Appendix II of the previously cited Corps of Engineers Engineer Manual EM 1110-2-1906 "Laboratory Soils Testing." Dry unit weights from laboratory measurements are given in Table 4 since the property is measured in the procedure for determining permeability.

Table 3

Size Gradation and Atterberg Limits

Field Sample No.	Depth ft From	Depth ft To	Laboratory Description (Classification)	Mechanical Analysis, % Finer: Gravel 2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index
			Site 4													
2	7.3	8.3	Sandy clay (CL)	--	--	--	--	--	100	99	96	88	77	64	34	20
3	12.4	13.5	Sandy clay (CL)	--	--	--	--	100	99	98	95	86	72	52	31	16
4	29.1	30.2	Sandy clay (CL)	--	--	--	--	--	--	100	96	87	76	62	38	24
5	33.3	34.5	Sandy clay (CL)	--	--	--	--	--	--	100	96	90	80	65	35	21
6	36.0	37.1	Sandy clay (CL)	--	--	--	--	--	--	100	99	91	75	57	32	15
7	41.4	42.4	Sandy clay (CL)	--	--	--	--	--	--	100	99	96	83	62	33	16
8	53.1	54.2	Clay (CL)	--	--	--	--	--	100	99	95	94	91	89	46	28
			Site 7													
2	9.6	10.9	Sandy clay (CL)	--	--	--	--	--	100	98	93	84	70	58	49	32
3	12.2	13.3	Sandy clay (CL)	--	--	--	--	--	--	--	100	99	96	85	45	27
4	17.4	18.7	Clayey sand (SC)	--	--	--	--	100	99	96	80	67	49	37	35	18
5	27.9	29.0	Clay (CL)	--	--	--	--	--	--	--	100	98	96	89	45	26
6	38.1	39.1	Sandy clay (CL)	--	--	--	--	--	--	100	99	94	85	71	35	20
7	40.5	41.5	Clayey sand (SC)	--	--	--	--	--	100	99	92	73	50	34	30	13
8	45.5	46.5	Sandy clay (CL)	--	--	--	--	--	100	96	91	90	87	77	42	21
			Site 9 (Watchtower)													
3	12.3	13.6	Sandy silty gravel (GM)	100	85	77	72	65	52	44	36	33	31	27	47	19

(Continued)

Table 3 (Concluded)

Field Sample No.	Depth, ft From	Depth, ft To	Laboratory Description (Classification)	Gravel 2 in.	Gravel 1 in.	Gravel 3/4 in.	Gravel 1/2 in.	Gravel 3/8 in.	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index
						Site 9 (Downslope)										
2	15.2	16.3	Clay (CH)	--	--	--	--	--	--	--	--	100	99	97	61	38
3	16.5	17.5	Clay (CH)	--	--	--	--	--	--	--	--	100	98	96	65	40
5	20.9	24.0	Clay (CL)	--	--	--	--	--	--	--	100	99	97	90	45	21
10	44.7	45.9	Silty clay (CL)	--	--	--	--	--	--	--	100	99	99	95	42	18
11	48.6	49.8	Silty sand (SM)	--	--	--	--	--	--	--	100	99	82	50		NP*
15	80.0	81.2	Sandy clay (CL)	--	--	--	--	100	99	99	96	93	86	66	34	14
17	91.3	92.3	Clayey gravelly sand (SC)	--	100	99	96	92	84	66	37	26	19	14	39	16
						Site 10										
6	10.0	11.2	Sandy clay (CL)	--	--	--	--	--	--	100	99	92	83	66	30	14
J-4	13.3	13.6	Sandy clay (CL)	--	--	--	--	--	--	100	98	93	84	65	30	14
10	18.6	19.5	Sandy clay (CL)	--	--	--	--	100	99	99	96	90	83	72	43	26
J-7	24.2	24.5	Sandy clay (CL)	--	--	--	--	--	--	100	98	94	88	77	41	24
12	25.8	27.0	Sandy clay (CL)	--	--	--	--	--	--	100	96	90	81	67	34	19
J-8	28.5	29.0	Sandy clay (CL)	--	--	--	--	--	100	99	93	84	76	65	38	21
15	32.5	34.9	Clayey sand (SC-SM)	--	--	--	--	--	100	97	86	70	52	32	24	4
16	39.7	42.1	Clayey sand (SC-SM)	--	--	--	--	--	--	100	96	79	51	27	25	5
J-11	41.0	41.4	Clayey sand (SC)	--	--	--	--	100	99	96	84	67	51	38	--	--
17	45.7	48.7	Clay (CH)	--	--	--	--	--	--	100	99	98	97	90	52	32
18	56.2	57.3	Silty sand (SM)	--	--	--	--	--	--	--	100	96	55	13	--	NP

* NP = nonplastic.

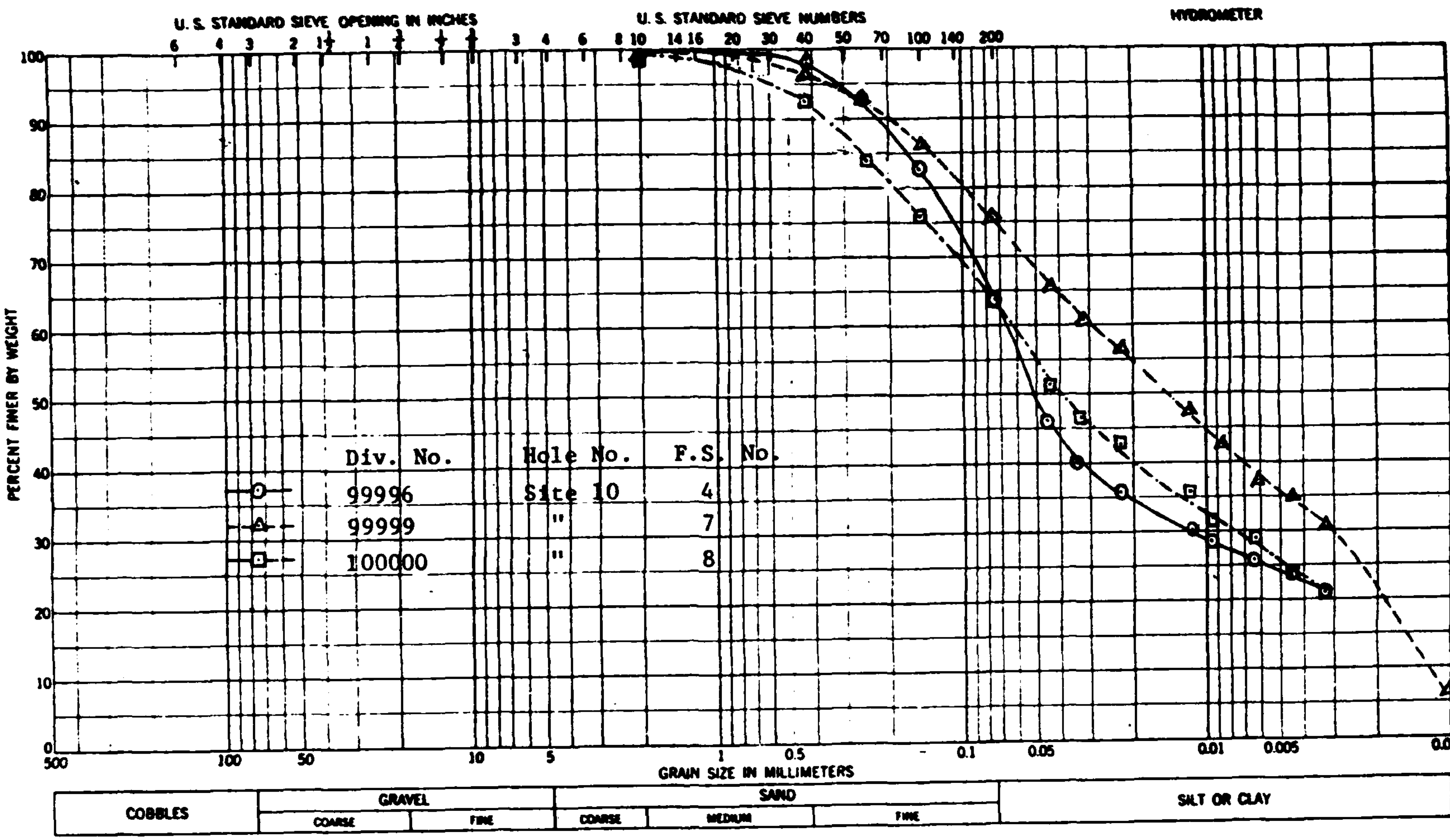

Figure 7. Grain sizes from sieve and hydrometer analyses of jar samples J-4, J-7, and J-8 at Site 10

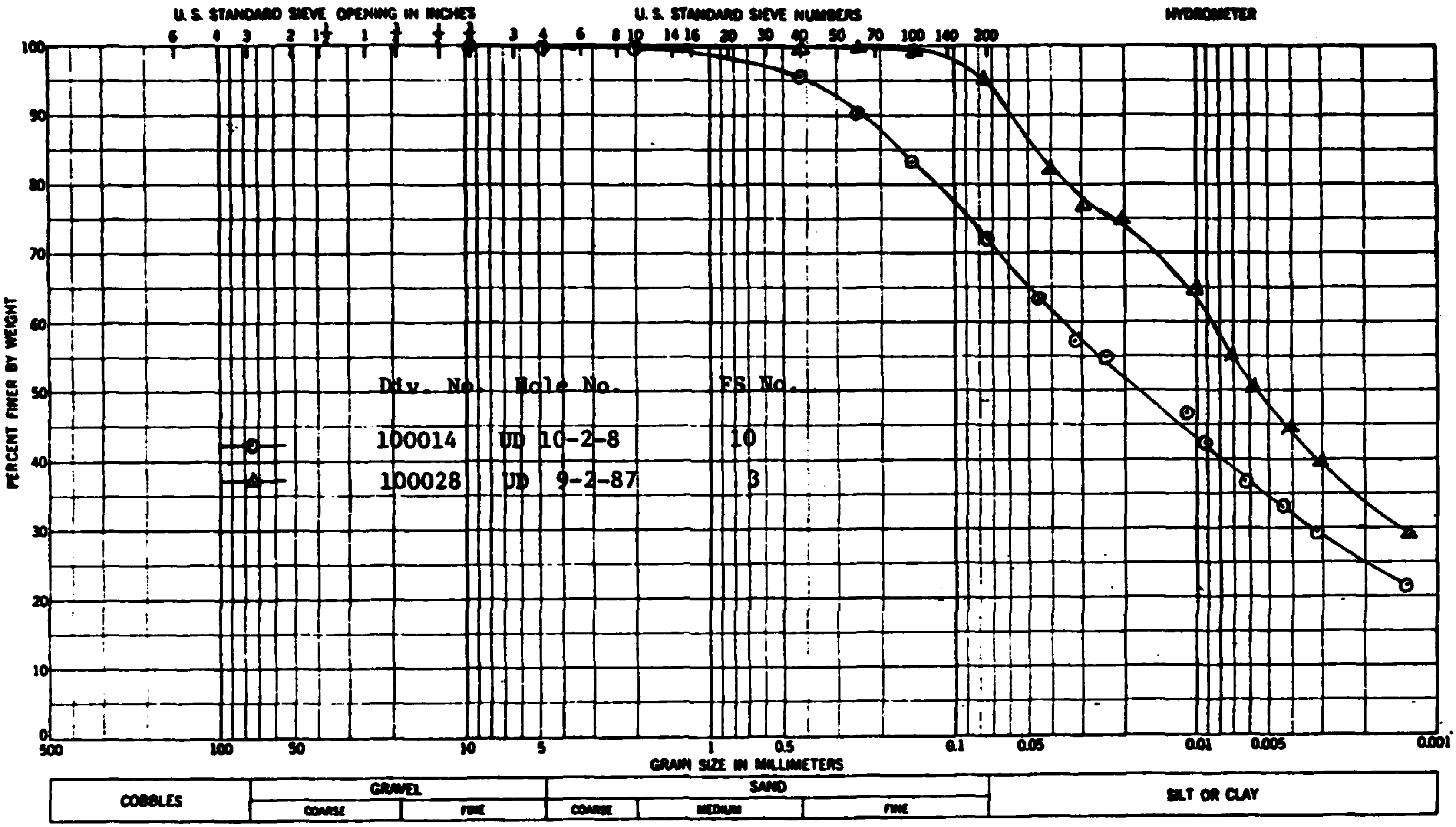

Figure 8. Grain sizes from sieve and hydrometer analyses of samples No. 10 at Site 10 and No. 3 at Site 9

Coefficient of Permeability

61. Two apparatus were available to measure the coefficient of permeability k in the laboratory. A triaxial cell was used in most tests, but a larger test cell with lower pressure capacity was used on those few coarsely sandy soils where high pressure was not deemed necessary.

Orientation

62. All tests except one used the vertical orientation and thus gave vertical k values across stratification. The exception at Site 10 was oriented horizontally to minimize the conspicuous effects of vertical root channels. Bedding at NWS Concord is usually indistinct at the small scale of laboratory specimens so vertical and horizontal k values can usually be considered equivalent.

Backpressured tests

63. These tests were generally conducted according to the procedures described in Appendix VII of Engineer Manual EM 1110-2-1906. An undisturbed cylindrical soil specimen is trimmed from the core sample to 2.8-in. diam and 2-in. height. The specimen is encased in a rubber membrane and subjected to an external hydrostatic pressure within a triaxial cell. The flexible boundary prevents leakage along the side of the specimen which would occur if the specimen were tested in a rigid permeameter.

64. Gas bubbles in the pores (unsaturated) can invalidate the general results. Therefore, the technique of backpressuring was used to reduce bubble volume by forcing gas into solution. To each degree of saturation, short of complete saturation a corresponding pressure exists sufficient to achieve full saturation. The chamber utilizes equipment that permits increasing external pressure and pore pressure simultaneously, maintaining a constant pressure difference. The components and system for testing are shown schematically in Figure 9.

65. To achieve saturation, the following steps were followed:

- a. Determine the dimensions and wet weight of the test specimen.

- b. Encase specimen in membrane, and place in center of the pressure chamber taking care to avoid entrapping any air beneath the membrane or along the base or cap. Fill chamber completely with pressure fluid.

Table 4

Permeability from Laboratory Tests

Field Sample No.	Depth, ft From	Depth, ft To	Dry Unit Weight pcf	Coefficient of Permeability cm/sec	Remarks
				Site 4	
2	7.3	8.3	110.9	3.28×10^{-6}	
3	12.4	13.5	115.0	2.87×10^{-6}	
4	29.1	30.2	115.0	1.54×10^{-7}	
5	33.3	34.5	109.7	6.07×10^{-5}	
6	36.0	37.1	110.7	3.72×10^{-6}	
7	41.4	42.4	105.4	1.39×10^{-6}	
8	53.1	54.2	106.2	7.55×10^{-9}	
				Site 7	
2	9.6	10.9	108.7	2.08×10^{-6}	
3	12.2	13.3	103.6	1.05×10^{-7}	
4	17.4	18.7	106.0	4.48×10^{-5}	Low-pressure cell.
5	27.9	29.0	108.5	2.52×10^{-8}	
6	38.1	39.1	111.1	6.85×10^{-6}	
7	40.5	41.5	111.9	1.64×10^{-5}	Low-pressure cell.
8	45.5	46.5	103.0	8.26×10^{-7}	
				Site 9 (Watchtower)	
3	12.3	13.6	115.6	--	Granular soil; not tested.
				Site 9 (Downslope)	
2	15.2	16.3	87.8	2.45×10^{-7}	
3	16.5	17.5	82.5	4.85×10^{-6}	
5	20.9	24.0	94.9	8.57×10^{-5}	See Part VI on crack permeability.
10	44.7	45.9	93.5	1.23×10^{-7}	
11	48.6	49.8	88.5	2.17×10^{-4}	
15	80.0	81.2	102.5	3.96×10^{-6}	Low-pressure cell.
17	91.3	92.3	113.9	1.09×10^{-5}	Low-pressure cell.

(Continued)

Table 4 (Concluded)

Field Sample No.	Depth, ft From	Depth, ft To	Dry Unit Weight pcf	Coefficient of Permeability cm/sec	Remarks
				Site 10	
6	10.0	11.2	91.3	1.41×10^{-4}	Horizontal orientation.
10	18.6	19.5	103.1	2.41×10^{-8}	
12	25.8	27.0	109.8	1.36×10^{-6}	
15	32.5	34.9	--	--	Disturbed; not tested.
16	39.7	42.1	97.1	3.24×10^{-4}	Low-pressure cell.
17	45.7	48.7	98.7	1.30×10^{-7}	
18	56.2	57.3	93.1	4.13×10^{-3}	Low-pressure cell.

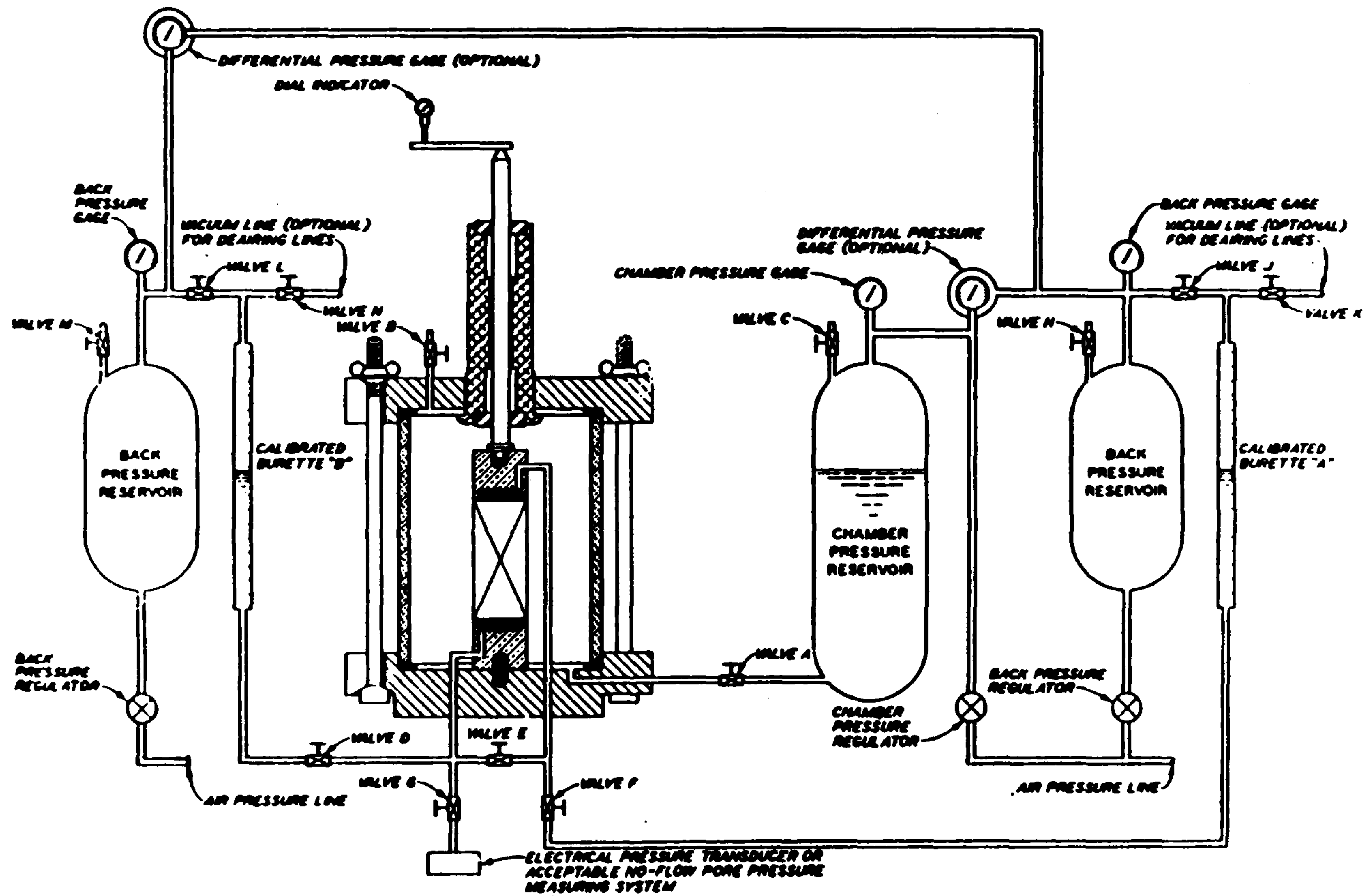

Figure 9. Apparatus for permeability determination in laboratory

c. Saturate the drainage lines and porous inserts in the base and cap with deaired water.

d. Apply backpressure in small increments with sufficient time between increments to permit equalization of pore-water pressure throughout the specimen. The difference between chamber pressure and back pressure is not allowed to exceed 5 psi.

e. Adjust the pressure regulators to a chamber pressure of about 7 psi and a back pressure of about 2 psi while keeping valves closed.

f. Open valve A to apply the preset pressure to the chamber fluid and simultaneously open valve F to apply the back pressure through the specimen cap. Immediately open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially constant, close valves F and G and record the burette reading.

g. Using the same technique, increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 5 psi less than the chamber pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure ranges from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, the pore-pressure reading equal to the applied back pressure immediately upon opening valve G. Back pressure of about 100 psi was needed for saturation in these tests.

h. Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen is considered saturated when the increase in pore pressure immediately equals the increase in chamber pressure.

After saturation was verified, the test proceeded as follows:

a. With the drainage valves closed, increase the chamber pressure to attain the desired effective consolidation pressure (chamber pressure minus back pressure). In these tests effective pressures ranged from 0.5 to 2.5 tsf, mostly to simulate the original depth of the specimen in the field. At zero elapsed time, open valves E and F.

b. Record time, dial indicator reading, and burette reading at elapsed times such as 0, 15, and 30 sec, 1, 2, 4, 8, and 15 min, and 1, 2, 4, and 8 hr. Dial readings and burette readings are plotted on an arithmetic scale versus elapsed time on a log scale. When the consolidation curves indicate that primary consolidation is complete, valves E and F are closed.

c. Apply a pressure to burette B greater than that in burette A. The difference between these pressures corresponds to the head loss h; h divided by the height of the specimen after consolidation L is the hydraulic gradient. The difference between the two pressures is kept as small as practicable, consistent with the requirement that the rate of flow be large enough to make accurate measurements of the quantity of flow within a reasonable period of time. Head loss h was always less than 20 cm. in these tests.

d. Open valves D and F. Record the burette readings at zero elapsed time, and make readings of burettes A and B and of temperature at various elapsed times. Plot arithmetically the change in readings of both burettes versus time. Continue making readings until the two curves become parallel and straight over a sufficient length of time to determine accurately the rate of flow (slope of the curves).

e. Determine the wet and dry weights of the specimen.

66. The computation of the coefficient of permeability k_{20} in centimetre per second at 20°C is as follows:

$$k_{20} = \frac{QLR_t}{hAt}$$

where

Q = quantity of flow, cm^3

L = length of specimen over which head loss is measured, cm

R_t = temperature correction factor for viscosity of water

h = loss of head in length, L, cm

A = cross-sectional area of specimen, cm^2

t = elapsed time, sec

The k values computed from the test results are given in Table 4.

Low-pressure tests

67. The triaxial cell system is unsuitable for testing relatively permeable soils because of limitations in flux through the porous inserts in the specimen base and cap. A larger pressure chamber was used on permeable soils. Test specimens were trimmed from core samples to 4-in. diam and 2-in. height.

The specimen was encased in a rubber membrane, confined in the chamber, and tested similarly to the procedures for the triaxial cell except that the chamber was limited to low confining pressure as well as low back pressures. These low-pressure test results are distinguished separately in Table 4.

PART VI: SITE EVALUATIONS

68. Five sites passing the site screening (PART III) were evaluated through subsurface and other investigations Sites (4, 7, 8, 9, and 10). The evaluations of individual sites are presented below. To facilitate comparison, the following distinctions are made somewhat arbitrarily for the relative permeability of soils.

<u>a</u>. low-permeability soils with $k \leq 10^{-6}$ cm/sec

<u>b</u>. medium-permeability soils with $10^{-6} < k < 10^{-4}$ cm/sec

<u>c</u>. high-permeability soils with $k \geq 10^{-4}$ cm/sec

This grouping is intended to distinguish categories present in roughly similar abundance. A two-fold grouping into soils satisfying and soils failing the State criterion of $k \leq 10^{-7}$ cm/sec would be less revealing since relatively few samples were tested, and among these, few had k values satisfying the criterion.

<u>Site 4</u>

69. Site 4 is located at the edge of Clayton Valley northeast of Mount Diablo Creek (Figure 3) and covers an area of about 15 acres. Ground surface slopes uniformly westward at 5 percent. Situated between Los Medanos Hills and Mount Diablo Creek, the site can be presumed to include clay-rich alluvium from the nearby hills and more sorted, pebbly alluvium from the upstream areas of Mount Diablo Creek. This presumed interlayering was not confirmed among the cores recovered from the center of the site, although those layers with basaltic pebbles are suggestive of a source on the diabasic mass exposed on Mount Zion 6 miles to the south.

70. The log of strata to the full depth of 58.0 ft is given in Figure 10. The water table is at 42 ft. The results of physical tests and laboratory and field permeability tests are presented in Tables 1 through 4. Summation of the characteristics of media is presented in Figure 11 where logging details and laboratory and field test results are generalized to an approximate representation of the site. According to this generalization and inherent interpretations, the strata are 0 percent low-permeability soils, 56 percent medium-permeability soils, and 44 percent high-permeability soils to a depth 15 ft below the water table. Only one layer among those tested was

DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		CLAY (CL), silty, sandy	100		
			80		
		4.5 SILT (ML), sandy clayey	100	1	
		6.9 CLAY (CL), sandy, silty, cohesive, w/ pebbles	75	2	
10			100		
			100	3	
		14.9 GRAVEL (GM), sandy w/ clay	100		
		15.3 SILT + CLAY (ML-CL) w/ gravel	100		
20		18.3 SAND w/ abundant gravel (SW-GW), loose; gravel fragments as large as 5"	0		Drilled 20.7 - 26.4 w/ rotary rockbit. Drill went down fairly rapidly but w/ gravelly action.
			0		
			30		
		28.5 CLAY (CL) sandy w/ pebbles	100	4	
30		30.2 GRAVEL (GC) clayey	100		
		30.7 CLAY (CL) silty, sandy, cohesive, w/ scattered small pebbles, w/ numerous caliche veins	100	5	
			85	6	
40			100		
			100	7	
		43.5 GRAVEL (GM) and sandy CLAY (CL), interbedded	100		Drilled 44.5 - 53.7 w/ rotary rockbit. Drill action alternated gravelly and smooth
50			0		
		53.9 CLAY (CH) dense stiff	90	8	
		55.7 GRAVEL (GP), loose, uniform	100		
		57.1 CLAY (CH), dense, stiff			
		58.0 TOTAL DEPTH 58.0 FT			

Figure 10. Log of drill hole at Site 4

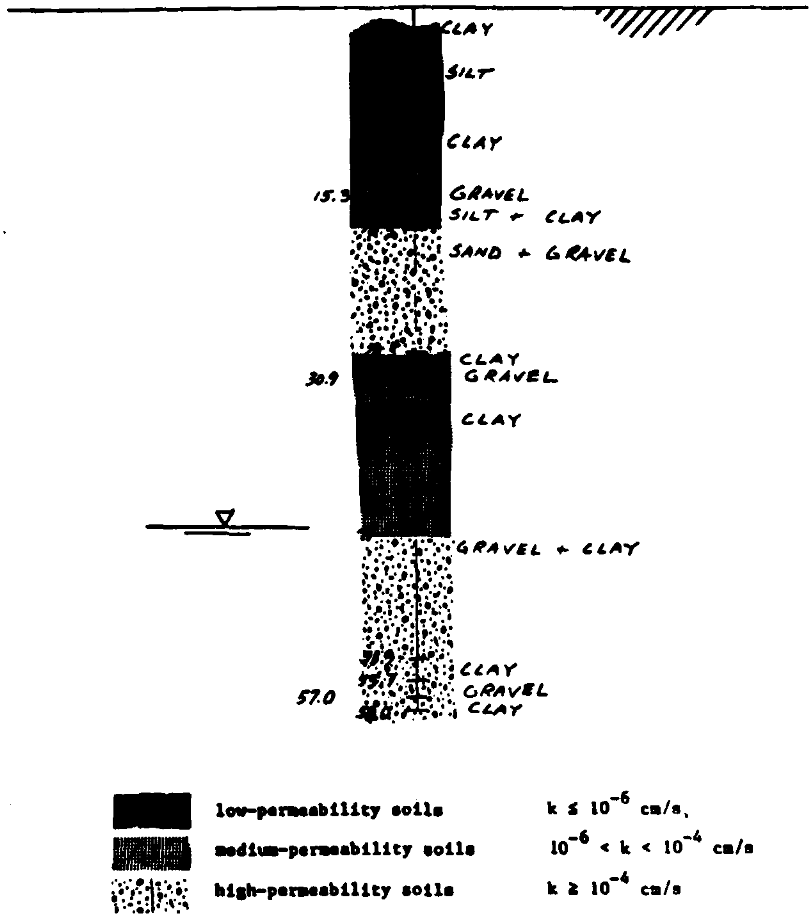

Figure 11. Generalization of Site 4 at drill-hole location

found with k value close to satisfying the pertinent State criterion (PART II), i.e. $k \le 10^{-7}$ cm/sec. This layer at 28.5 to 30.2 ft is represented by sample No. 4 in Table 4. The top 4 ft of gumbo soil at the site was not tested but probably satisfies the State criterion also, except for root holes.

71. Low-density, well-sorted, granular soils were not found among samples within 50 ft of the surface, but such material presenting a potential for liquefaction during earthquake shaking may possibly remain hidden in the unsamples granular interval between 18 and 28 ft. The Clayton fault passes about 500 ft from the nearest side of the site, not close enough to violate the State criterion on Holocene faults.

72. Landslides are not a threat on the relatively flat valley floor. The site is not in the track of any reasonably hypothetical mudflow from Los Medanos Hills. The usual foundation settlements that would occur during the period when waste is disposed can be accommodated easily within design and

construction efforts pursuant to the development of a disposal site. Cumulative settlement will be relatively small.

73. The suggested flood line for a 100-year event is considered to be well down slope from Site 4. This conclusion is based on information obtained from the Federal Emergency Management Agency (FEMA) that the 100-year flood will rise to about 9 ft above the bed of Mount Diablo Creek. Figures 12 and 13 show similar flood profiles immediately adjacent to NWS Concord with 100-year heights at about 9 ft above the bed of Mount Diablo Creek.

Site 7

74. Site 7 covers an area of about 15 acres southwest of Mount Diablo Creek and south and adjacent to Willow Pass Road (Figure 4). Ground surface slopes northeastward at 5 to 10 percent. The site is located partly in the floodplain of Mount Diablo Creek but mostly on the flank of one of a group of broad hills trending northwest to the southwest of Mount Diablo Creek, hence the relatively steep upper slope. The broad hills are considered to be older alluvium, somewhat firmer than that alluvium situated centrally to Clayton Valley. The sources of old and young alluvium are presumed to be in the upper reaches of Mount Diablo Creek to the southeast since basaltic pebbles were logged. No demarcation between the Mount Diablo Creek alluvium and old alluvium was recognized to the depth drilled, but such a change would probably be very subtle and difficult or impossible to pinpoint.

75. The log of strata to the full depth of 46.2 ft is presented in Figure 14. The water table is at 34 ft. The results of physical tests and laboratory and field permeability tests are presented in Tables 1 through 4. Summation of the characteristics of media is presented in Figure 15 where logging details and laboratory and field test results are generalized to an approximate representation of the site. As generalized, the strata are 31 percent low-permeability soils, 69 percent medium-permeability soils, and 0 percent high-permeability soils to 10 ft below the water table. Only two layers among those few tested appear to meet the State criterion for permeability (i.e. $k \leq 10^{-7}$ cm/sec). These layers are represented by samples No. 3 and 5 in Table 4.

76. The site appears to lack low-density, well sorted, granular soils within 50 ft of the ground surface and therefore appears to have low potential

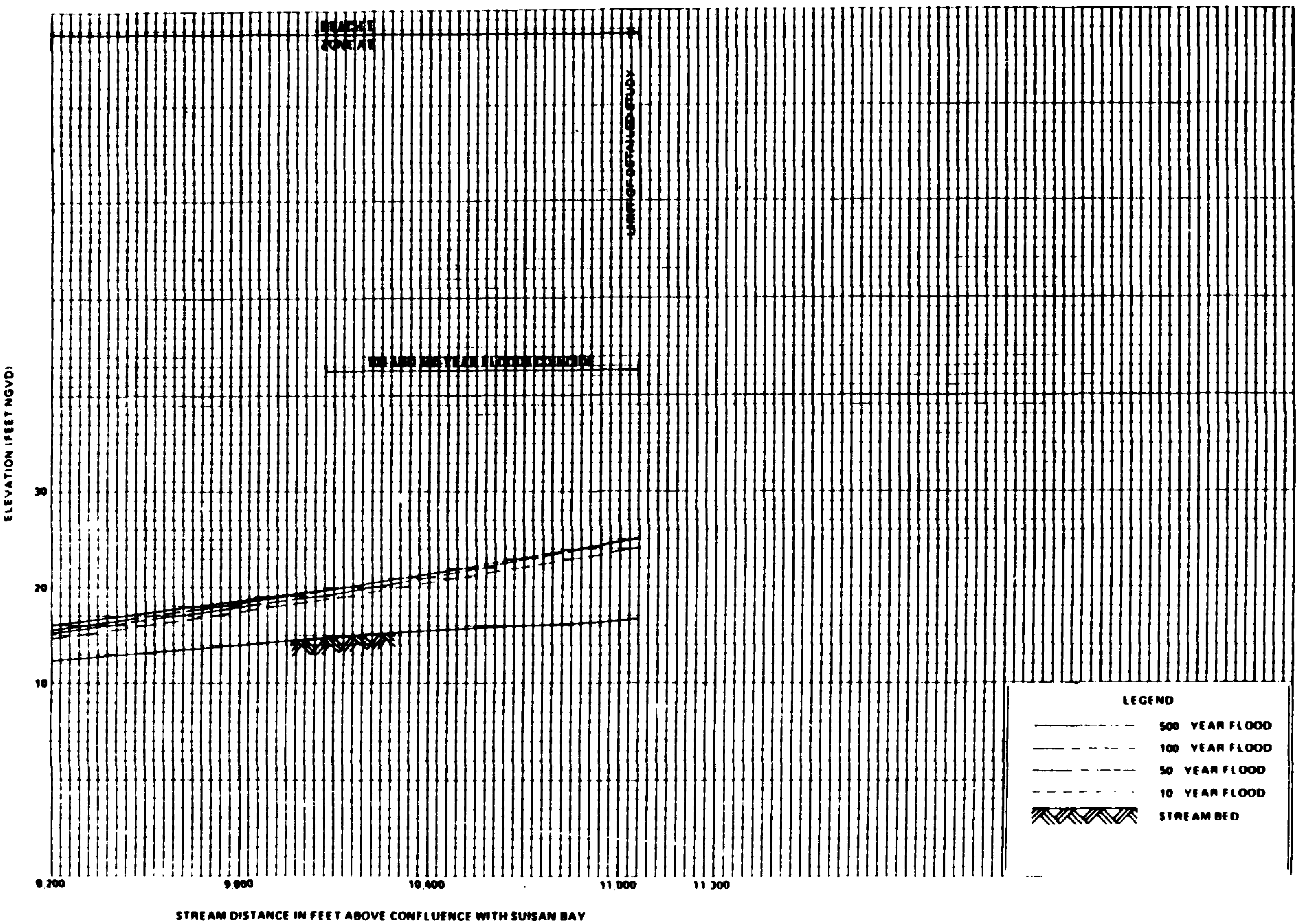

Figure 12. Flood profiles for Mount Diablo Creek downstream from NWS (FEMA)

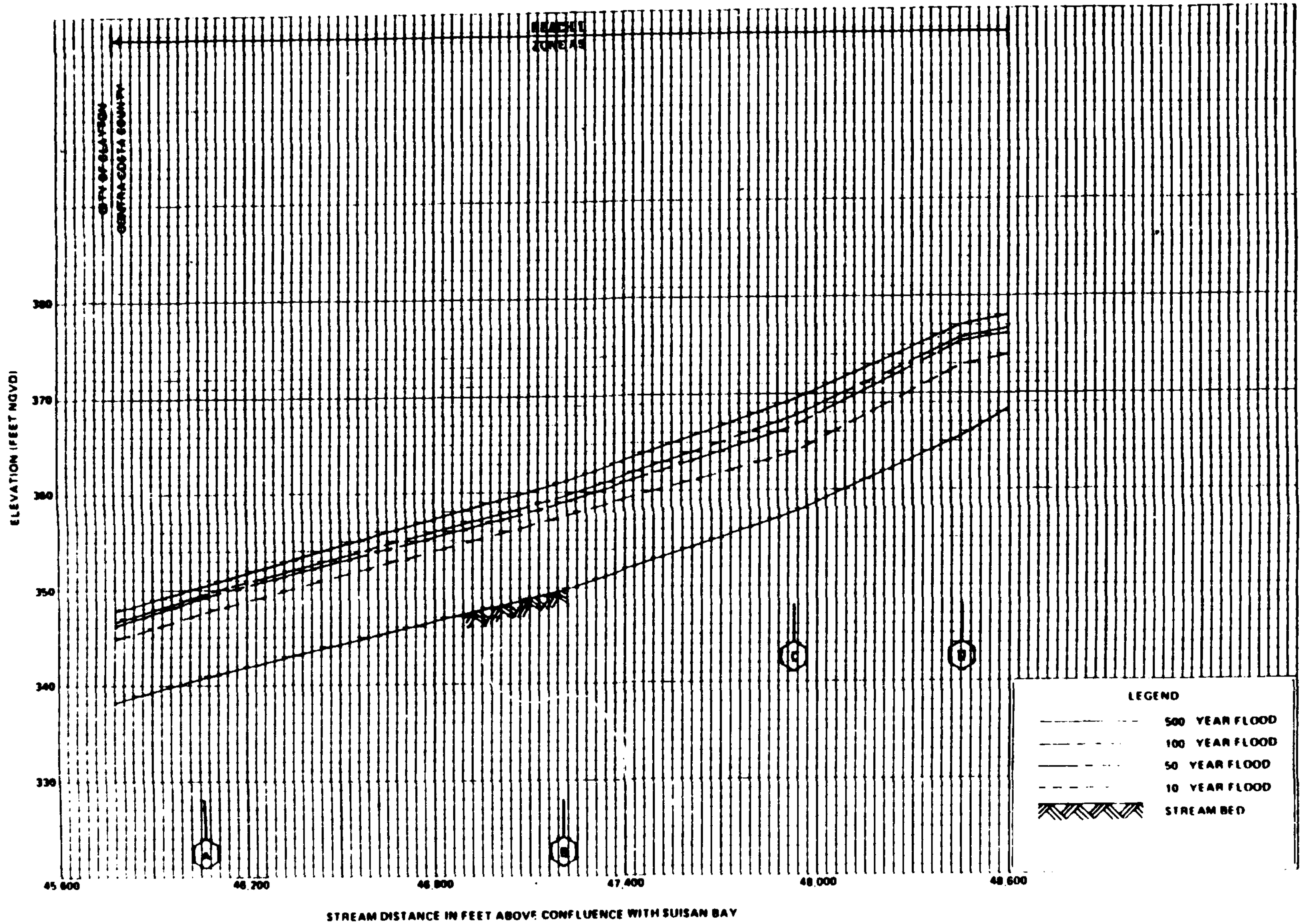

Figure 13. Flood profiles for Mount Diablo Creek upstream from NWS (FEMA)

DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		CLAY, silty (CL-ML), sandy, w/ some gravel, w/ rootlets above 3.0'	100		
			100	1	
		7.2	100		
		GRAVEL (GM), sandy	100		
		9.6	0		Drilled 7.9-9.6 w/ rotary rockbit
10		CLAY (CL), silty; w/ sand + gravel below 13.3	100	2	
			80	3	
		15.9	100		
		SAND (SM-SC), silty w/ clay	100		
			100	4	
20			25		Poor recovery & disturbed core caused by large gravel fall-in pieces in hole, 19.4-24.7
		24.7	25		
		CLAY (CL) silty, reddish brown w/ streaks of gray stiff clay and organic specks	100		
		29.5	100	5	
30		SILT (ML-CL), clayey			
		31.5	100		
		CLAY (CL), silty, w/ caliche veins & nodules and organic specks	100		
			100		
		39.1	100	6	
40		SAND (SC), clayey w/ organic specks			
		41.4	100	7	
		CLAY (CL), silty, w/ caliche veins & nodules, organic specks, streaks of stiff gray clay	100		
		46.2	100	8	
		TOTAL DEPTH 46.2 FT			

Figure 14. Log of drill hole at Site 7

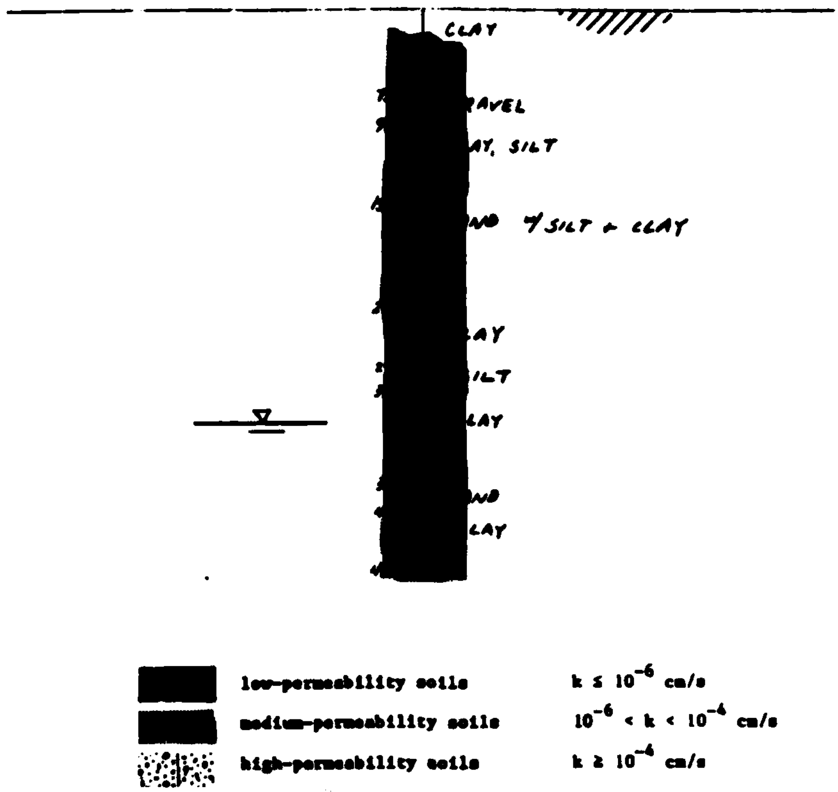

Figure 15. Generalization of *Site* 7 at drill-hole location

for liquefaction during earthquake shaking. It should be recognized however, that core recovery was poor at depth 19 to 25 ft (Figure 14), and susceptible beds might conceivably be present but undetected. This remote possibility should be pursued in the future if this site is selected for further investigations. Landsliding is unlikely on the relatively gentle valley floor. The low position of the water table also disfavors liquefaction and landsliding. Settlements would occur as the waste is disposed on the site material but would be relatively minor and can be accommodated in design and construction of such a facility.

77. The suggested flood line for a 100-year event is considered to lie well down slope from Site 7. This conclusion is based on the information in Figures 12 and 13 from FEMA. Those figures show similar flood profiles for Mount Diablo Creek immediately upstream and downstream from NWS Concord with a 100-year height at about 9 ft above the steambed.

Site 8

78. Site 8 is located southwest of Mount Diablo Creek north and adjacent to Willow Pass Road (Figure 4) and covers about 26 acres. The site is about 1/2 mile northwest of Site 7. Ground surface slopes northeastward at 6 percent. The site is situated much like Site 7 on the flank of one of a chain of broad hills of old, firm alluvium. It seems likely that the gravelly alluvium which makes this site appear unsuitable as a disposal site is shallow alluvium related to Mount Diablo Creek. The gravelly alluvium can be considered to overlap the older, firmer alluvium of the broad hills.

79. One hole was drilled at Site 8 and passed through clayey silt near the surface and entered a gravelly layer at about 7 ft. The gravel was loose and could not be sampled. Drilling was stopped at 15 ft. Drilling did not reach the water table. No testing was undertaken since the soils were considered too permeable and unsuitable as site media for waste disposal. Figure 16 is a profile of Site 8 at the drill-hole location as far as it was

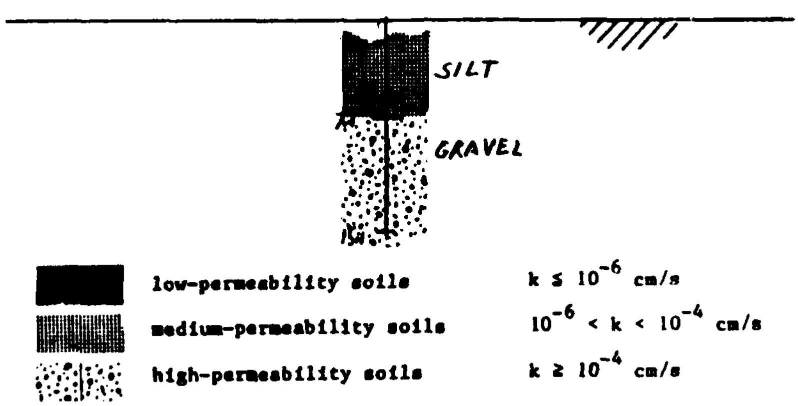

Figure 16. Generalization of Site 8 at drill-hole location

explored. Conceivably, suitable strata might be encountered elsewhere on site, particularly upslope to the southwest.

Site 9

80. Site 9 is situated high on the southwestern flank of the broad hills centrally situated in Clayton Valley (Figure 4). Other sites on the same group of hills are 6, 7, and 8. Ground surface is steeper than at the

other sites surviving screening but still averages only 13 percent southwestward across the area of about 40 acres. The soil at Site 9 is alluvium which is somewhat firmer and presumably older than alluvium at lower levels in Clayton Valley. Nevertheless, the alluvium at this location is probably from the same general sources along the headwaters of Mount Diablo Creek.

81. Alluvium at Site 9 is old enough to have experienced tectonic deformation, presumably in relation to movement on the Concord fault and others. An anticline has been mapped (Dibblee 1980c) as following the general northwest trend of the broad hills (Figure 1). Poor exposures of strata 1,000 ft to the south and nearby on Willow Pass Road suggest inconclusively a dip of about 7 deg to the southwest. Stratification was not found in the deep through cut for the Contra Costa Canal about 1,000 ft north of the drill holes.

82. The log of strata to the full depth of 110.0 ft is given in Figure 17. Ground-water level is documented in Table 2. The piezometric surface for the tip at 105 ft is at 48 ft. A shallower piezometer at 50 ft showed the water table at 47 ft and falling in March. This falling water table may have been reflecting shallow, seasonal effects dampened out at 105 ft. From this interpretation one might expect ground water to fluctuate intermittently between depths of 48 and 45 ft or even less.

83. The results of physical tests and laboratory and field permeability tests are given in Tables 1, 3, and 4. Summation of the characteristics of media are given in Figure 18 where geological details and test results are generalized to an approximate representation of the site. Accordingly, the strata are 0 percent low-permeability soils (as defined above), 100 percent medium-permeability soils, and 0 percent high-permeability soils to 40 ft below the water table. Only one layer among those tested came close to satisfying the k criterion in State requirements. This layer is represented by sample No. 10 in Table 4.

84. Site 9 was initially drilled at the watch tower on the crest of the hill. This hole encountered predominantly gravelly strata (Figure 19) before the location was abandoned in favor of the location downslope some 500 ft to the southwest. It was estimated that the crestal area is underlain by gravel and will be unsuitable for disposal purposes. This unsuitable portion of the site substantially reduces the usable area.

DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		CLAY (CH), fat, w/ rock fragments	100		
		3.8	65		
		SILT (ML), gray w/ brown + black mottling	0		4.8 - 7.2 sample lost due to worn drilling shoe
			90		
10		12.0	90	1	
		SILT, clayey (ML-CL), w/ orange & some black mottling	0		12.0 - 14.4 sample lost due to malfunction of core barrel
			100	2	
			100	3	
20			100	4	
			100		
			100	5	
		27.2	100		
		27.5 CLAY (CH), fat	100		
30		SILT, clayey (ML-CL) w/ common orange mottling + abundant slickensides	100	6	
			100	7	
			100	8	
		38.0	100		
40		SILTSTONE, blue, mod. hard; w/ blue silty clay @ top (38.0 to 38.6)	100	9	
		42.7	100		
		SILT, clayey and CLAY, silty (ML-CL), blue	100	10	
		47.5	90		
50		SAND (SM), fine, silty, dense 50.0	100	11	
		SILT (ML), clayey, blue, dense, sandy in part	100	12 13	
		54.8	60		
60		SILT, clayey, blue, and SILTSTONE (interpreted from drill cuttings)	0		Drilled 54.8 - 65.0 w/ rotary rockbit
			0		
		SILT, blue, clayey, w/ organic odor	100	14	
70 80			0		Drilled 69.8 - 80.0 w/ rotary rockbit

Figure 17. Log of drill hole at final location at Site 9 (Continued)

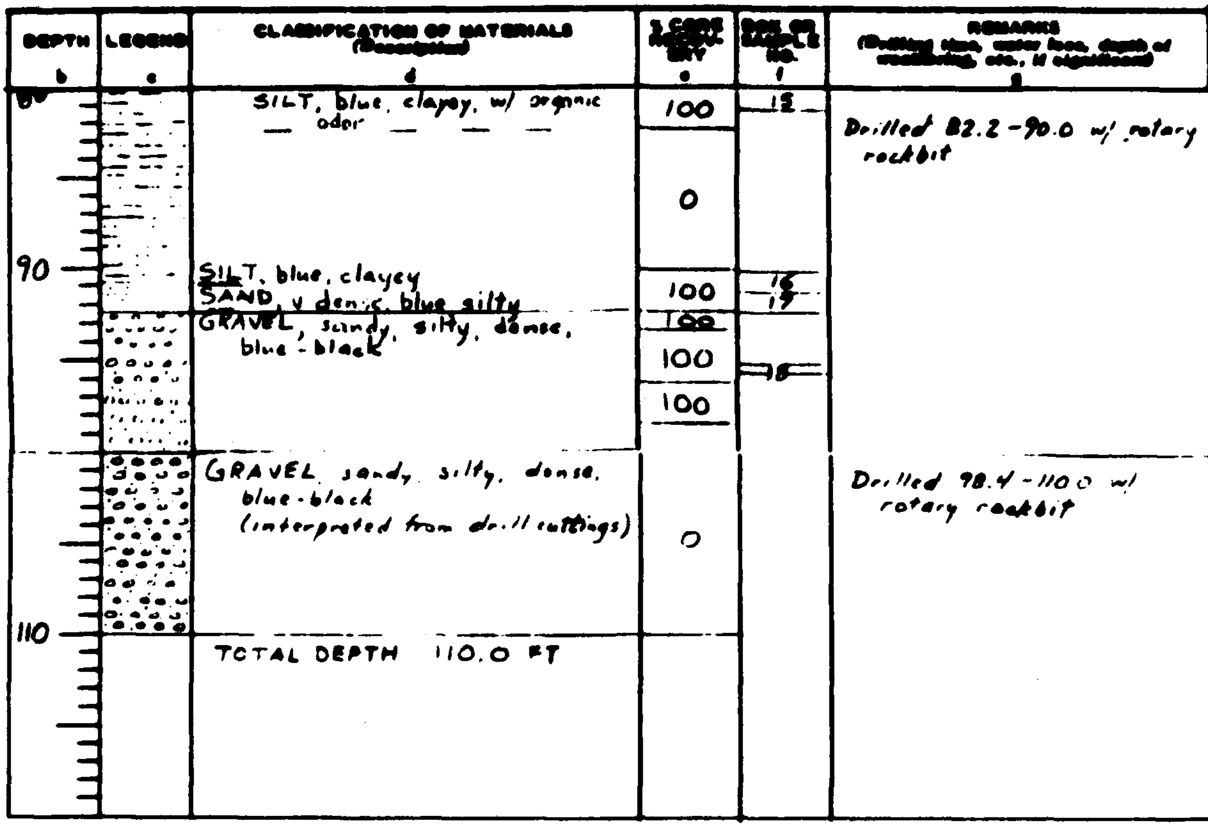

Figure 17. (Concluded)

85. Another manifestation of the apparently greater age of alluvium at Site 9 was a noticeable presence of hairline cracks in core samples. Some cracks appeared soon after extrusion from the sampling tube as though freed from heavy confinement of past overconsolidation. Some cracks were slickensided. The potentially detrimental effect of cracks on permeability of the soil media was demonstrated in the laboratory testing. Sample No. 5 had to be subjected to confining pressure substantially exceeding that equivalent to the field overburden to close cracks that had opened during specimen preparation (Figure 20). The k values decreased fivefold to 1.66×10^{-5} cm/sec as cell pressure was increased to 3 tsf.

86. A mineralogical examination was made in search of expansive clay minerals using the powder method of x-ray diffraction analysis with glyceration to simulate the introduction of planar water. Analyses on samples No. 3 and 10 both revealed a predominance of montmorillonite and quartz, with plagioclase feldspar and chlorite also present.

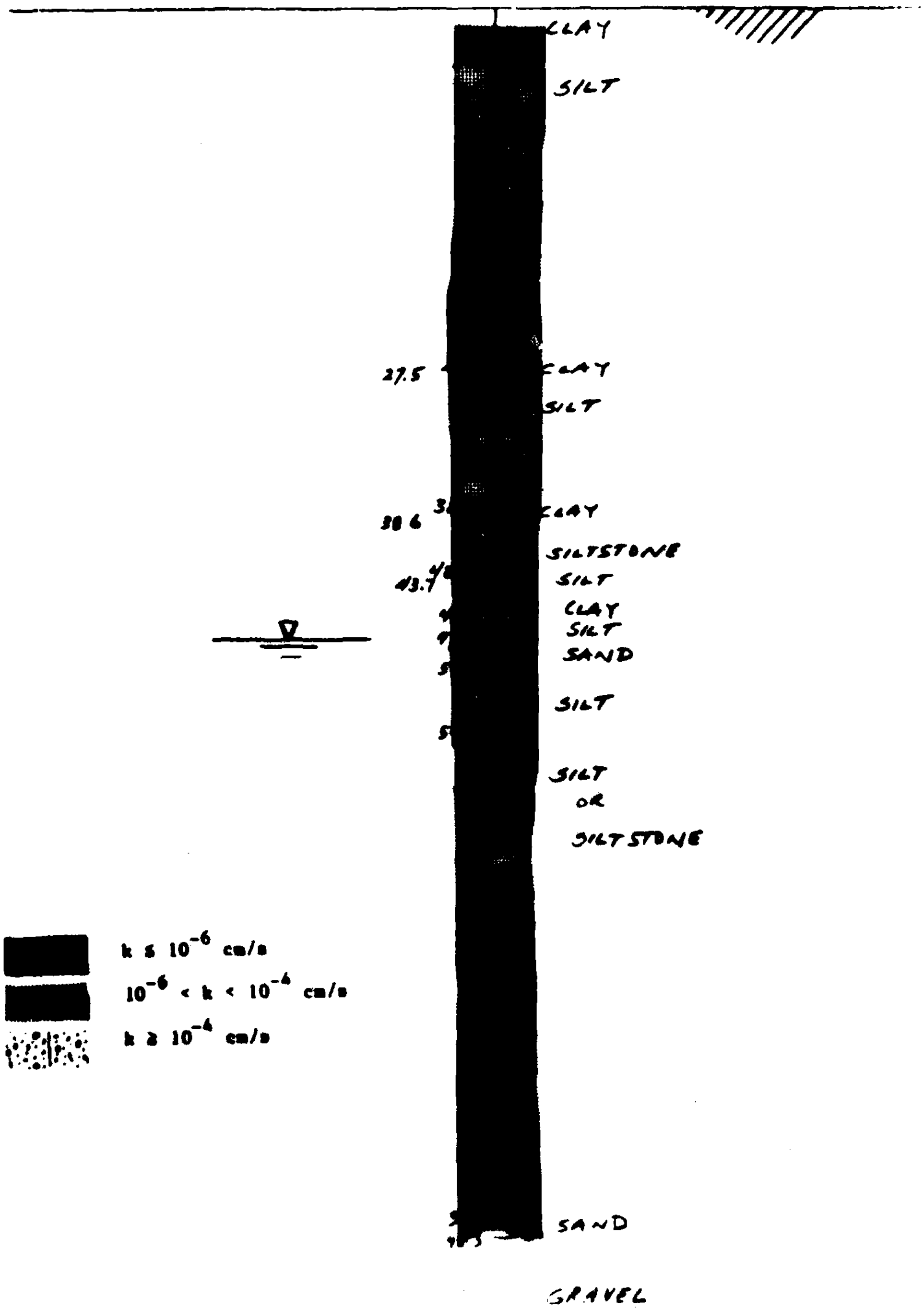

Figure 18. Generalization of Site 9 at final drill-hole location

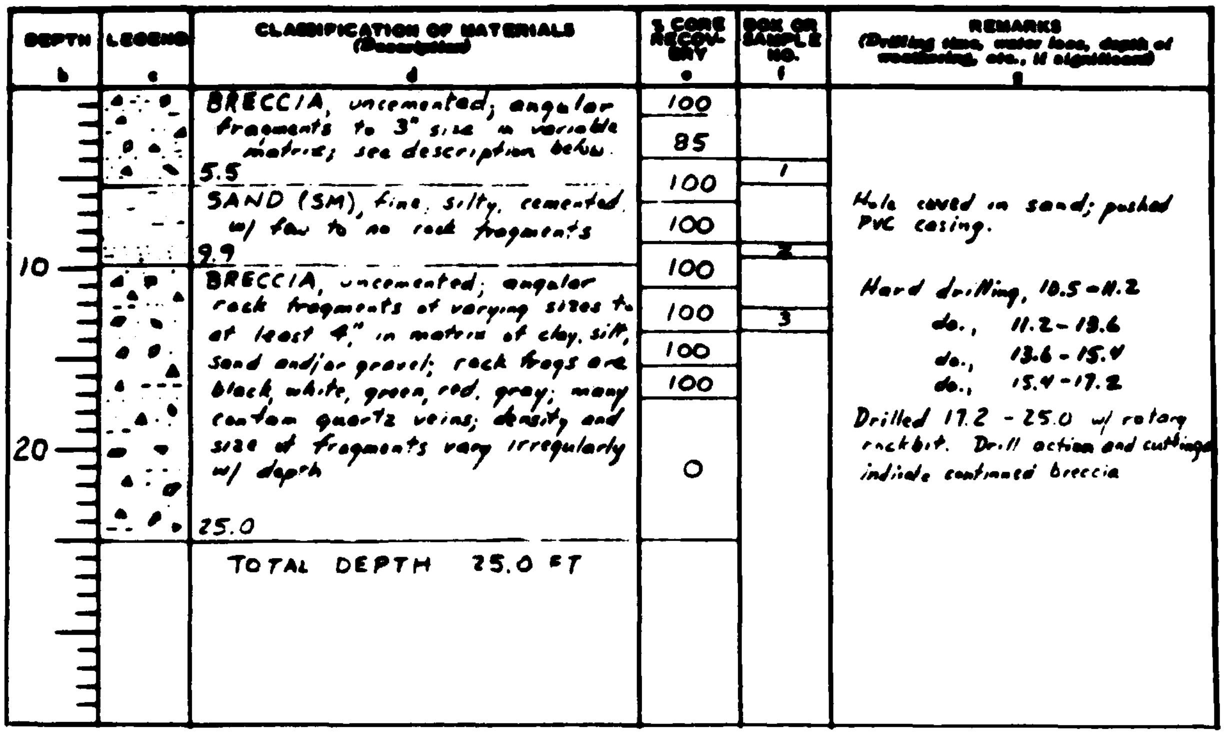

Figure 19. Log of drill hole at watchtower location at Site 9

Site 10

87. Site 10 is located in Section 5, T2N, R1W on the so-called Pittsburg Plain and midway between the north face of Los Medanos Hills and the wetlands of Suisun Bay (Figure 5). The area is about 36 acres, but most of it lies closer than 2,000 ft to private property and occupied buildings on the east. The lack of a 2,000-ft buffer would be a necessary sacrifice to obtain any advantages offered by the site. One advantage is proximity to the areas of contaminated soil targeted for possible disposal at the chosen site. Several areas of heavy-metal contamination are clustered 500 to 3,000 ft northeast of the drill-hole location, a short trucking distance.

88. The ground slopes 3 percent across a large alluvial fan accumulated northward from the mouth of a canyon in the hills. This alluvial fan dominates the site. Clay-rich strata down to about 49 ft are probably derived from the hills. At 49 ft the strata change to more sandy nature and contain conspicuous mica flakes. The mica-bearing sandy soil is most likely a product of riverine processes in the valley of the Sacramento - San Joaquin system and from a source in the Sierra Nevada. An interlayering of fan and riverine alluvium probably continues to much greater depth.

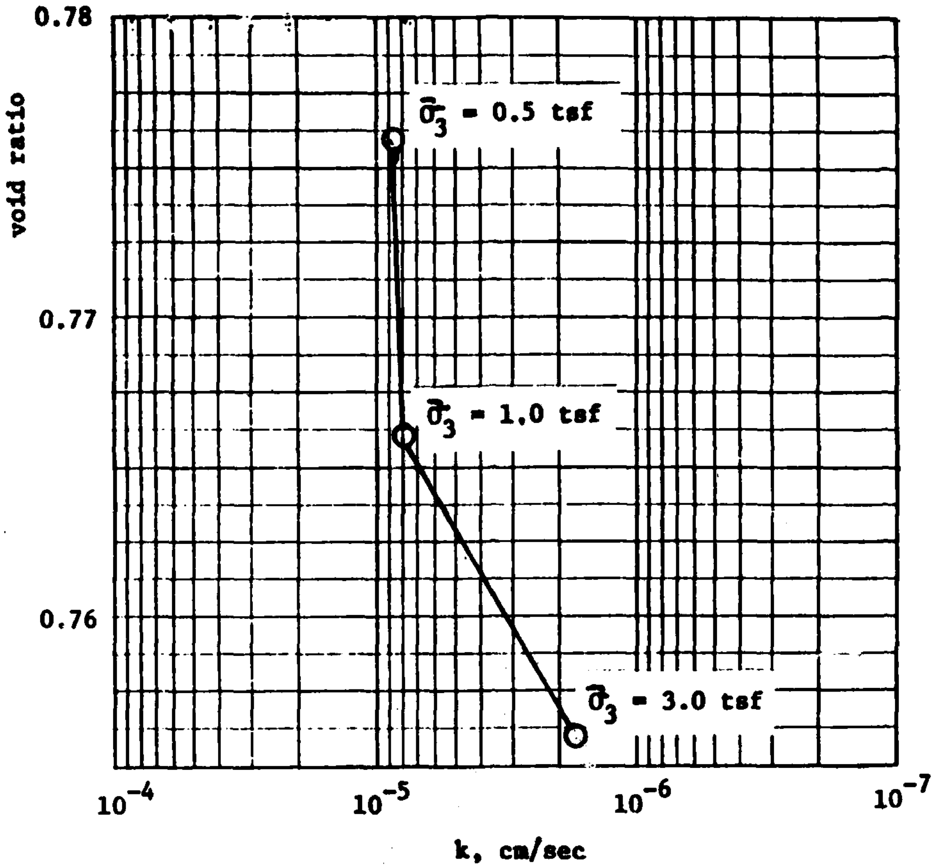

Figure 20. Decrease in coefficient of permeability with increase in cell pressure on sample No. 5 from Site 9

89. The log of soils to the full depth of 61.0 ft drilled at Site 10 is given in Figure 21. Veinlets of chalky calcite occur in the clay-rich layers. Apparently some drying and hairline cracking has affected these layers after they accumulated in a presumably semiarid environment. The spacing of these incipient cracks averages as small as 0.8 in. in some layers. Samples tested in the laboratory included such features to ensure that k values would be inclusive of possible secondary permeability where these potentially important defects are present.

90. The results of physical tests and of field and laboratory permeability tests are presented in Tables 1 through 4. These results are integrated with the geological log to summarize the media as regards permeability (Figure 22). The soils are 24 percent of low permeability (as defined above),

DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		CLAY (CH) fat, organic, silty below 2.4	100		
		4.8	100	1	
		SILT (ML), clayey, sandy, w/ some gravel, w/ sand seams 5.2-5.3, 8.3-8.4, sand/gravel seam 7.5-7.7	85	2, 3	
			100	4, 5	
10			90	6, 7	
			85	8	
		15.9	70	9	
		GRAVEL, sand + silt (GM)	0		
		19.6 SILT (ML) clayey w/ gravel	100	10	
20		20.7 CLAY, silt, sand + gravel (GC)	100	11	Hard drilling 20.5-22.9
		24.3	100		
		SILT (ML), clayey, w/ some gravel, w/ sand/gravel seam 31.2-31.3, sand seams 32.2-32.3, 32.5-33.7	85	12	
30			100		
			100	13, 14	
			100	15	
		36.8	100		
		CLAY and SILT (CL-ML) w/ some gravel + sand seams	100		
40			100	16	
			0		Drilled 42.1 - 47.5 w/ rotary rockbit
		48.1			
50		SAND (SM), silty, dense, w/ mica	100	17	Hard drilling 47.5-49.9
			0		Drilled 49.9-55.0 w/ rotary rockbit
			100		
60		61.0	0		Drilled 57.4-61.0 w/ rotary rockbit
		TOTAL DEPTH 61.0 FT			

Figure 21. Log of drill hole at Site 10

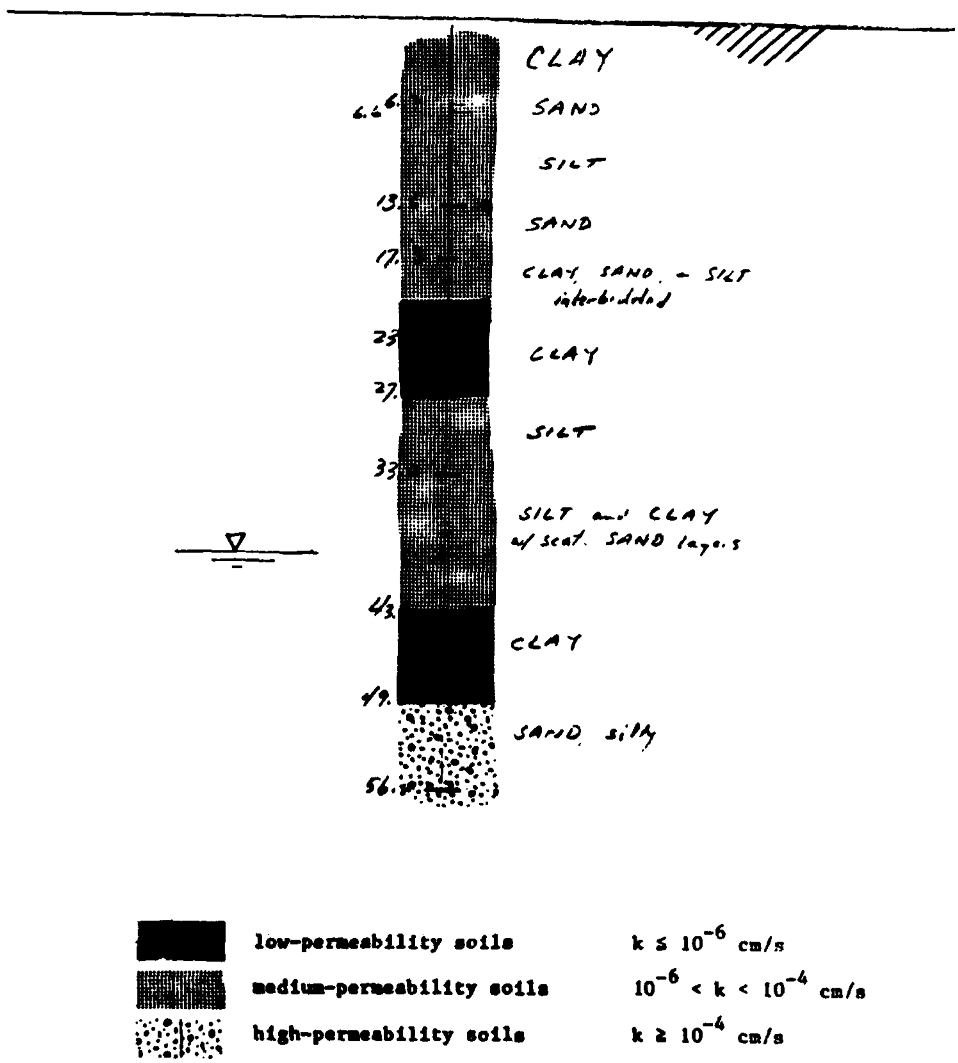

Figure 22. Generalization of Site 10 at drill-hole location

64 percent of medium-permeability, and 12 percent of high-permeability. Only one layer among those tested met the State criterion of $k \leq 10^{-7}$ cm/sec. A second layer was close. These layers are represented by samples No. 10 and 17 in Table 4.

91. The water table at Site 10 is at 39 ft below the surface (Table 2). This is approximately 6 ft above sea level and the level of water in Suisun Bay. Ground-water flow direction was not determined in this study, but the indicated head difference is compatible with a northward flow toward the bay.

92. The site is regarded as safe from threats of tsunami waves or 100-year flood by its elevated location. The absence of well-sorted granular layers above 49 ft depth precludes shallow liquefaction. The great depth of the micaceous sandy soil below 49 ft makes liquefaction unlikely.

PART VII: CONCLUSIONS

93. Eleven sites were considered as candidate locations for a Class I hazardous waste disposal facility at the Naval Weapons Station, Concord, Calif. Investigations and evaluations were formulated with emphasis on criteria defined by the State of California.

94. None of the sites investigated met all of the acceptance criteria. The extent to which the sites fell short of these criteria are discussed below. It was also appropriate to explore the consequences of the deficiencies and the extent to which the effects of these deficiencies might be circumvented by an engineered solution. Accordingly, engineering alternatives are considered below to provide guidance for future investigations or actions.

Suitability by Criteria

95. Six of the eleven candidate sites survived screening based on surficial requirements in State criteria. Five of these six surviving sites were investigated by drilling, sampling, and testing and were evaluated further for suitability according to the permeability of the site media. These sites are ranked below in estimated order of most suitable to least suitable for isolation of waste from ground water.

Site Number	Location	Area Acres	Formation	Feet to Water Table
9	Inland	40	Older alluvium	48
10	Tidal	36	Fan alluvium	39
7	Inland	15	Older alluvium	34
6	Inland	13	Older alluvium	35 estimated
4	Inland	15	Fan alluvium	42
8	Inland	26	Mount Diablo alluvium on older alluvium	35 estimated

Site 6 was not drilled but is considered to be similar to Site 7 located nearby.

96. Even the sites considered nearest to suitable (Sites 9 and 10) fail to meet the State criterion requiring $k \le 10^{-7}$ cm/sec for sufficient thickness to preclude migration of contaminants to the ground water. This

conclusion is not firmly fixed since "sufficient" is undefined in the State guidance as regards assumptions and the analytical basis for determining thickness. From a very conservative point of view, a thickness of several tens of feet might be perceived as appropriate since any sustained rainfall, cell leakage, or other liquid input at the ground surface would saturate the media and allow a continuing flux advancing at up to 10^{-7} cm/sec. Regardless of viewpoint, great thickness helps to counteract percolation to the extent that intermittent interruptions in liquid input (as from seasonal and droughty periods) lead to desaturation and a reversal in movement. Thus, water may descend under conditions of large input at the surface but later rise when input is interrupted and contravening processes such as capillary rise occur. In this way the thickness of low-permeability strata delineates a buffer zone in which the intermittent descent and rise of water from the surface are harmlessly confined.

97. Of some additional concern is the fact that the few layers of lowest permeability found at the sites are randomly positioned. Ideally, the least permeable stratum should be situated at the top immediately beneath the waste unit. Accordingly, even the few layers which meet the permeability criterion may be somewhat ineffective by virtue of their remote position with respect to the waste unit.

98. Random positioning will also greatly complicate the design and construction of the disposal facility. Optimum grade lines for cut and fill or other site preparation cannot be expected to correspond to the top of the most favorable stratum; therefore, a tradeoff will have to be faced. Additional subsurface investigations to develop the three-dimensional geological picture would be necessary before any site could be considered much further as a candidate for waste disposal.

Engineering Alternatives

99. Reliance on engineering to upgrade a marginal or unsuitable site to suitable status is one approach which may deserve consideration. Probably the most promising design feature for offsetting the shortcomings of the natural sites at NWS Concord would be the construction of a thick foundation of compacted, clay-rich soil. In this way the designer could gain a measure of control on the properties and construction not achievable by using only the

natural foundation. Sources of clay-rich soil for such construction may be available on and near some sites. The burden of demonstrating effectiveness of engineered features however would lie with the designer and represents a difficult task.

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